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Innovations for Navigation Projects Research Program

Underwater Geotechnical Foundations

Landris T. Lee, Jr., and Richard W. Peterson

December 2001

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**Innovations for Navigation
Projects Research Program**

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Underwater Geotechnical Foundations

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Preface

This report was prepared for Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Innovations for Navigation Projects (INP) Research Program. The study was conducted under INP Work Unit 33144, "State of the Art for Preparing Foundations Underwater," managed at the U.S. Army Engineer Research and Development Center (ERDC), Vicksburg, MS.

Dr. Tony C. Liu was the INP Coordinator at the Directorate of Research and Development, HQUSACE; Research Area Manager was Mr. Barry Holliday, HQUSACE; and Program Monitors were Messrs. Bruce Riley and Mike Kidby, HQUSACE. Mr. William H. McAnally of the ERDC Coastal and Hydraulics Laboratory was the Lead Technical Director for Navigation Systems; Dr. Stanley C. Woodson, ERDC Geotechnical and Structures Laboratory (GSL), was the INP Program Manager.

The research and preparation of the report were accomplished by Dr. Richard W. Peterson, Principal Investigator for Work Unit 33144, and Mr. Landris T. Lee, Jr., Co-Investigator, both of the ERDC GSL. Work was conducted under the general supervision of Dr. Joseph P. Koester, Chief Geotechnical and Earthquake Engineering Branch; Dr. Robert L. Hall, Chief, Geosciences and Structures Division; and Dr. Michael J. O'Connor, Director, ERDC GSL.

At the time of publication of this report, Dr. James R. Houston was the Director of ERDC, and COL John W. Morris III, EN, was Commander and Executive Director.

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1 Introduction

The U.S. Army Corps of Engineers has been tasked with designing, building, and maintaining numerous structures located on or within bodies of water including rivers, lakes, waterways, and coastal areas. Each of these structures has a geotechnical foundation system that enables the structure to perform a useful service or function throughout its design life. Each foundation is uniquely designed to match the structure's function and engineering requirements constrained by the underwater geological site conditions.

Background

Underwater foundations have been constructed throughout history. The earliest type of underwater foundation appears to have been driven stakes or piles around the edges of water bodies. Ancient settlements built on piles around lakes in present-day Scotland, Switzerland, Italy, and Ireland have been dated to about 4,000 years ago. The ancient Phoenicians built docks and ports (such as Tyre) using underwater construction methods. The Greeks and Romans used piles for shore works along the Mediterranean and many other locations (Fleming et al. 1992, D'Appolonia 1972). Herodotus, a Greek writer who lived in the 4th century B.C., wrote about African dwellings erected on piles driven into a lake. In Britain, evidence of bridge timber piles about 9 ft (3 m) long was found in the Tyne River. Vitruvius, a Roman architect, wrote the treatise *De Architectura*, which describes using sheet piles for dams and other water structures. The Roman engineers also developed concrete and used it for placement of bridge piers. During the Middle Ages, cities such as Venice and Amsterdam were built upon timber piles (Fleming et al. 1992).

Not until the 19th century did soil engineering principles become incorporated into the foundation construction process. Changes in materials and installation technology began to take place. Pile-driving by man or horse power was replaced by a steam engine that raised a cast-iron ram and released it to impact the timber pile. Metal piles became available in the mid-1830s. In 1824 Joseph Aspdin patented his hydraulic cement, which became known as portland cement, and the French introduced reinforced concrete. Driven piles could be replaced by bored shafts since the hydraulic cement could be placed underwater (Fleming et al. 1992).

Modern soil mechanics, which offered an explanation for the behavior of soil foundations, was not advanced until the early 20th century. Since that time, numerous foundation types, materials, and installation technologies have been developed for onshore applications (Parkhill 1998). Underwater foundation technology initially lagged behind onshore technology, especially in soil sampling and testing (Focht and Kraft 1977). In other areas such as site assessment, foundation material selection, installation methods, and equipment, the offshore technology has been uniquely developed. For example, the offshore industry almost exclusively used pile foundations without benefit of preliminary site investigations until the 1940s (Focht and Kraft 1977). Design guidelines for some types of underwater foundations have evolved separately from onshore guidelines (American Petroleum Institute 1993), while site assessment methods and installation equipment have been specifically developed for underwater foundations.

Deepwater site exploration and foundation construction techniques have been used for scientific research and military purposes, but the offshore industry has a profit motive to build functional foundations (Brown 1972). Although the underwater foundation projects designed and constructed by the U.S. Army Corps of Engineers may not be located beyond the Continental Shelf, the use of state-of-the-art offshore foundation applications will likely be influential.

Objectives

Underwater foundation selection and design choices have generally been based on foundation construction in the dry, that is, within a pre-installed cofferdam that isolates the construction from the surrounding body of water, or on dry land with water diversion. Construction on dry land is normally the fastest and least expensive method (American Society of Civil Engineers (ASCE) 1998). However, the dry land method may not be an option unless the body of water can be feasibly diverted. Cofferdam design and construction efforts require additional time and expense to be budgeted for project completion. The benefits of faster paced and more economical construction within the water (in the wet) without cofferdams have been demonstrated in many case histories, and the Corps has begun exploring the feasibility of in-the-wet technology, as illustrated in Figure 1. The Corps' first major in-the-wet project (Monongahela River Braddock Dam) is currently under construction and is expected to save \$5 to \$15 million while reducing contract duration by 1 year (ASCE 1999a).

To obtain a technical project database of in-the-wet foundation design and construction requires time and experience, and some Corps Districts are compiling individual project information at specific sites. No written Corps-wide guidance for underwater foundation design and construction is currently available. Most of the technical expertise is held by specialist design firms and contractors. The information contained in this report cannot be found in a concise form or single document elsewhere in the literature.

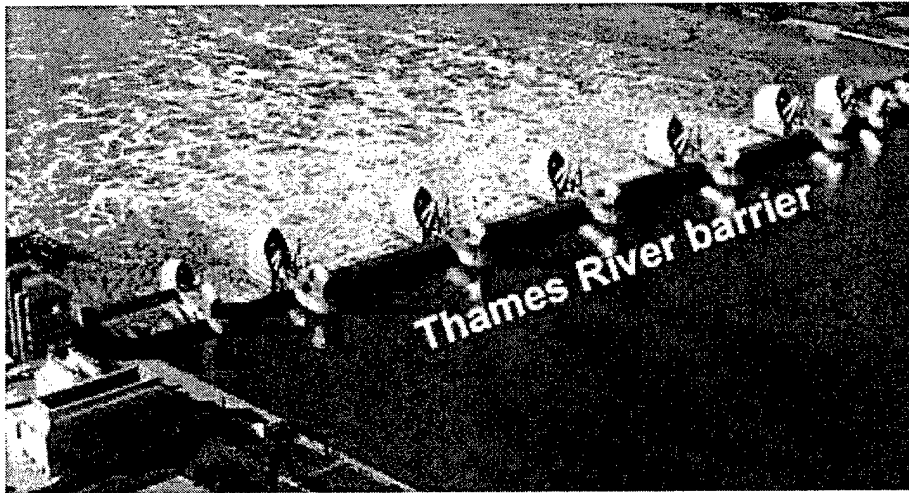


Figure 2. Thames River flood barrier gates (after Gilbert and Horner 1984)

The foundation project consisted of constructing four piers in sequence from the south side of the river, followed by construction of five piers in sequence from the north side. Access to the piers was provided by a jetty constructed from the south side. Bottom sills connected each pier. The riverbed consisted of chalk overlain with alluvial deposits of gravel, sand, and clay up to 50 ft (16 m) thick. Six pier foundations were sited on the chalk, and three pier foundations were sited on sand deposits. Underwater excavations and concrete placements were required (Figure 3). Prior to excavation, sheet piles forming a future cofferdam were driven into the chalk at each pier location. The distance from the tops of the sheet piles to the base of the excavation was about 90 ft (27 m).

Special problems were encountered during the underwater foundation preparation. Leveling and cleaning up the bottom of the excavation in the chalk posed a challenge due to redeposition of river silt during each tide. Explosives were used to remove chalk wedged in the sheet-pile troughs. Special air-lift pumps were developed to scour the excavation bottom.

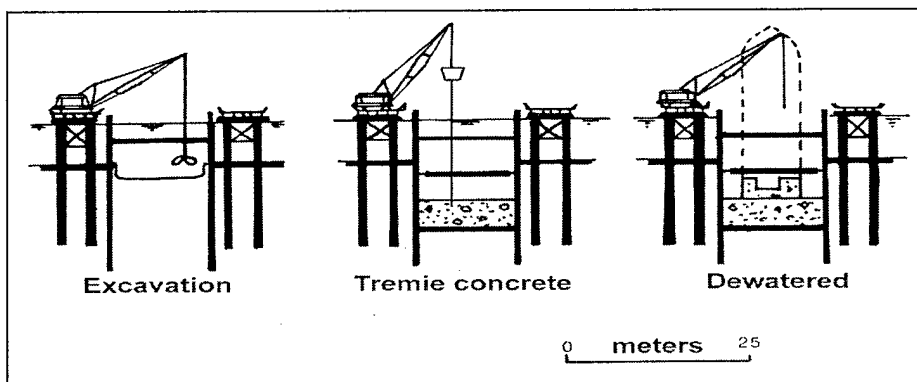


Figure 3. Underwater foundation construction sequence (after Gilbert and Horner 1984)

Underwater concrete placement for each pier base was accomplished using 12-in. (30-cm)-diam tremie pipes transporting approximately 7,000 tons of concrete around the clock for 5 days. Each concrete base thickness was 15 ft (5 m). Coring confirmed optimal jointing with the undisturbed chalk surface. The cofferdams were dewatered, and pier construction proceeded. By 1982 the project was essentially completed.

Eastern Scheldt storm surge barrier

On the eastern side of the English Channel, the Netherlands are also subjected to the whims of the North Sea. The 1953 floods provided an impetus for the Government to launch the Delta Project, which was designed to protect the Netherlands at the confluence of the Rhine, Maas, and Scheldt Rivers. After construction of seven dams and flood barriers, the Eastern Scheldt storm surge barrier and two auxiliary lock and dam structures (which comprised the final part of the project) were completed in 1986 (DOSBOUW 1987).

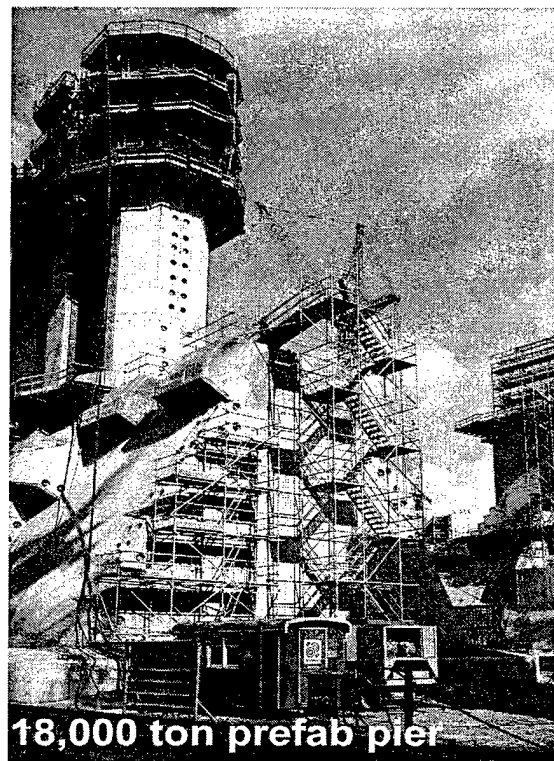
The storm surge barrier spans the mouth of the Eastern Scheldt, and was the most complex portion of the entire Delta Project. The typical current is about 5 fps (1.5 m/sec) and bottom slope is 1 in 7 (Heijnen and Vermeiden 1979). Three tidal channels with depths up to 99 ft (30 m) were crossed with a structure whose foundations consisted of piers and connecting sills. Each pier was prefabricated using prestressed concrete and weighed up to 18,000 tons. Figure 4 shows the barrier layout with a typical pier detail. The piers were built inside a dry ring dike with a bottom approximately 49 ft (15 m) below sea level. To move the piers, the dike was flooded, and each pier developed a buoyant weight of about 9,000 tons. A lifting vessel capable of hoisting 10,000 tons lifted and transported each pier to its final site. Once at the site, each pier was lowered to the prepared seabed, internally ballasted, and grouted at the prepared foundation contact.

The seabed floor consisted of young Holocene fine to medium sands and silt. It was modified to accept the piers. No piling or concrete foundations were used. Site investigation using a specially designed geotechnical platform yielded cone resistances ranging from 14.5 to 145 psf (0.7 to 70 kPa) (Heijnen and Vermeiden 1979). Since these resistances were below those required for adequate bearing capacity, special preparations were required to enable adequate bearing capacities.

First, the seabed was excavated (a "cunette" excavation), and unsuitable material (silt) was replaced with sand where necessary. The sand was compacted over a distance of 263 ft (80 m) around each pier by a specially built floating compacting rig. The rig drove four large vibrating rods into the subsurface and compacted layers up to 59 ft (18 m) thick. The compaction process took 3 years to complete. The quality control system included another specially built craft equipped with a conventional drill rig and a diving bell that conducted soil sampling and density measurements during the compaction process.



a. Storm surge barrier



b. Storm surge barrier float-in pier detail

Figure 4. Eastern Scheldt storm surge barrier (after DOSBOUW 1987)

The compacted seabed was then dredged, leveled to the designed depth, and covered with $658 \times 138 \times 1.2$ ft (200×42 m \times 36 cm) prefabricated foundation mattresses. The mattresses were fabricated offsite and transported to each location, where they were unrolled onto the seabed by specialized floating equipment. Each mattress consisted of graded aggregates sandwiched between reinforced support fabric. After the upper mattress was placed, a block mattress (containing concrete blocks of varying thicknesses) for leveling purposes was positioned. Each pier was then placed directly on its respective block mattress.

A graded stone aggregate sill was placed between each pier to stabilize the piers and prevent bottom scouring in the event a gate failed to close. A stone-depositing barge crane placed the rocks weighing up to 11 tons (10 metric tons) with densities up to 2.5 tons/cu yd (3 metric tons/m³) into their underwater locations. About 5.5 million tons of stone were placed into position over a period of about 2 years. After completion of the underwater sill, precast concrete sill beams were lowered into place to rest on the underwater sill. The remaining superstructure elements were then positioned and placed between the piers.

Mississippi River Lock and Dam 26 guidewall

The upstream guidewall at the Corps' Lock and Dam 26 replacement on the Mississippi River is founded on 118 caissons. Each caisson consists of a 6-ft (1.8-m)-diam open-ended pipe pile driven 51 ft (15.5 m) into sands and gravels in the river bottom, which is about 40 ft (12.2 m) below the river surface. After vibratory driving was completed, the piles were internally excavated under a positive pressure head to prevent blowout and then filled with concrete. A load test using eight 3-ft (0.9-m)-diam reaction piles driven to bedrock validated design assumptions (Demsky and Moore 1989).

Monongahela River gated dam

A recently contracted \$107 million [in-the-wet] project is the Pittsburgh District's new gated dam next to the existing Braddock Locks and Dam 2 on the Monongahela River in Pennsylvania (U.S. Army Corps of Engineers 1998). The underwater foundation system will consist of a dredged area with sheet-pile cutoff walls in the riverbed alluvium and concrete drilled shafts socketed into bedrock, upon which will rest two float-in precast concrete dam elements (Figure 5).

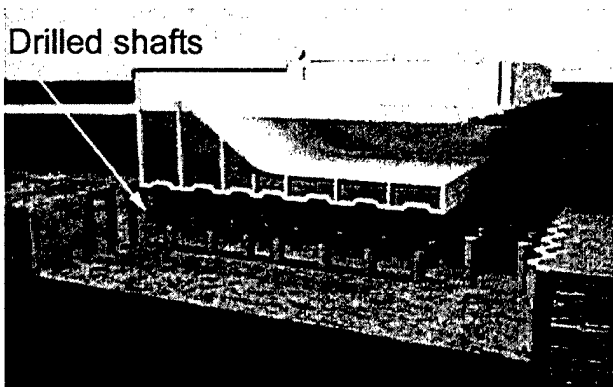


Figure 5. Monongahela River gated dam and foundation (after *Gerwick News* 1998)

Besides the novel features of the concrete float-in elements, the predominant work feature for the underwater foundation will be drilled shafts. These provide the deep foundation support for the precast concrete elements, and also function as anchor piles for positioning the precast concrete elements. The drilled shaft (similar to a caisson or pier) is basically a pinned foundation system consisting of a 78-in. (1.9-m)-diam by 30-ft (9-m)-long open steel pipe pile (casing) driven with a vibratory hammer to bedrock, then rotary-

drilled through the bottom end and further drilled without casing about 17 ft (5 m) into the bedrock. Steel reinforcement and instrumentation tubes for later sonic testing will be inserted to bedrock and tremie-concreted. Steel shear pin connectors provide the attachments for the subsequently positioned precast concrete dam elements.

The two 300-ft (91-m)-long, 103-ft (31-m)-wide, and 21-ft (6-m)-deep precast segments will be internally tremie-grouted. Once finished, the dam height will be about 81 ft (25 m) and about halfway submerged. Construction is expected to be completed in 2002 (ASCE 1999a).

Olmsted Dam

Another innovative Corps of Engineers project is being designed to replace and update navigation and flood control capabilities on the lower Ohio River, 15 miles from its confluence with the Mississippi River. The Olmsted Dam project has been designed as float-in (in-the-wet) construction and will be the only wicket dam in the United States when completed. Currently under contract are \$224-million twin locks being constructed within \$55-million in-the-dry cofferdams (*ENR* 1996).

Lake Mead intake structure

Lake Mead serves as the main source of drinking water for southern Nevada, including Las Vegas. The lake is impounded by the Hoover Dam. As part of a water supply improvement program, a water-intake shaft and tunnel system is being constructed. The intake shaft was installed in the wet and will eventually be connected to an underground tunnel and distribution system.

The intake structure was floated in and placed in 240-ft (73-m)-deep water onto a specially prepared underwater foundation on a steep underwater slope. A bench was excavated, and a pile-drilling template was positioned on the level excavation. Downhole hammer-drill equipment was used to drill anchor piles that were subsequently grouted. A second template was keyed into the first template, and a 20-ft (6.5-m)-diam shaft was drilled an average length of 100 ft (30 m) into stable rock. The first 30 ft (9 m) was steel-cased. The shaft was then grouted after the float-in steel riser intake structure was placed into position. Personnel dives were minimized during construction by using remotely operated vehicles (ROVs), a high-resolution positioning system, and sonar equipment (Norwesco Marine 1999).

Adriatic Sea piled breakwater structure

Approximately 1.25 miles (2 km) off the Italian coast near Manfredonia on the Adriatic Sea, a breakwater structure was constructed to protect new offshore docking facilities (Toppler, Harris, and Maaten 1971). The breakwater design was basically a heavy concrete slab resting on battered piles in 36 ft (11 m) of water. The 4-ft (1.2-m)-diam steel pipe piles are battered at a 3:1 incline to resist the design load of 3.3 tons/sq ft (30 tons/m²). A two-dimensional pile-driving template made of precast concrete was designed as a permanent addition to the breakwater structure, and extended above the waterline to serve as a driving guide. Model testing was conducted during the design phase.

Argentina piled jetty

The foundation of an offshore jetty installed at Puerto Deseado, Argentina, was constructed in the rhyolite bedrock (Nakayama 1992). Each of the 147 piers (diameter 4 ft (1.2 m)) was socketed 13 ft (4 m) into the rock using a

MACH-120R rock drill. The bedrock inclined approximately 30 deg, and drilling was conducted on a self-elevating (jack-up) platform in a water depth of 60 ft (18 m). The chosen rock drill operated by down-the-hole suspension hammering with 720 blows per minute, which helped eliminate slippage down the steep incline.

Offshore lighthouses

One of the first permanent offshore gravity-based structures is the Kish Bank Lighthouse located off the coast of Ireland (Young, Kraft, and Focht 1975). This structure was built in 1965 in the protected Dun Laoghaire harbor, floated 8 miles (13 km) off the entrance to Dublin Bay, and lowered 67 ft (20 m) to the seabed (Figure 6). Another gravity-based lighthouse is the Royal Sovereign Lighthouse, floated off the coast of Eastbourne, England, in 1968 and sunk in 47-ft (14-m) water depth (Antonakis 1972).

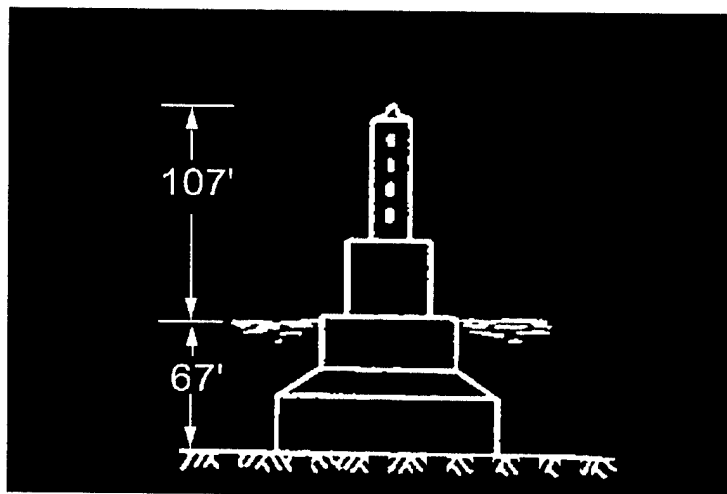


Figure 6. Kish Bank lighthouse—gravity-based structure (after Young, Kraft, and Focht 1975)

Hydroelectric dam

British Columbia's Hugh Keenleyside earth dam foundation was constructed in the wet on the Columbia River between 1965 and 1969 (Bazett and Foxall 1972). No seepage cutoff wall was designed since the bedrock is located about 450 ft (137 m) below the sand-gravel riverbed in 90 ft (27 m) of water. To control seepage, an impervious glacial till blanket was constructed beneath the future dam extending upstream 2,200 ft (669 m) from the future dam upstream toe. The as-designed maximum final slope of the blanket was 1:10.

The foundation was installed by free-fall dumping till, sand, and gravel materials into the river using bottom-opening barges and above-water bulldozers. The till blanket overlies sand and gravel backfilled in water depths to 55 ft (17 m).

Extensive survey control was established to define the boundaries and depth of the underwater foundation. An echo-sounding vessel defined the water depth and bottom topography. Shoreline-surveyed grid lines established the vessel's survey patterns using shore-based transits and hand-held radio communication. Current and turbidity meters were used to monitor the river's velocity and water quality. Turbidity exceeding 5 ppm was not allowed during construction because of adjacent industrial water-supply intakes.

Underwater inspection and sampling tasks were accomplished primarily by scuba divers wearing wetsuits. The divers monitored preliminary hydraulic dredging and subsequent construction operations for silt sediment control, which had a prominent influence on the project's schedule. Another major problem was segregation and loss of fines during dumping. Quality control measures included density determinations using standard penetration tests, gradation analysis sampling, large-scale testing experiments, and piezometer instrumentation.

Other dams

The High Aswan Dam in Egypt was partially constructed underwater in a maximum water depth of 131 ft (40 m). The underwater portion of the core consists of coarse sand that was grouted after placement. Vibrator-compacted dune sands form adjacent underwater zones. The dam height is 364 ft (111 m). Hong Kong's Plover Cove Main Dam was partially constructed underwater after soft foundation materials were removed. Other projects listed by Johnson, Compton, and Ling (1972) include the Mississippi River's Chain of Rocks rock-fill dam, Columbia River's Dalles Closure and Wanapum Dam, and Ghana's Akosombo Dam.

The first dam project built across a major U.S. river without channel diversion was the Chain of Rocks rock-fill dam across the Mississippi River near St. Louis, MO. This \$4.5-million Corps of Engineers (St. Louis District) project avoided the time and expense of diverting the river, building cofferdams, or dewatering the site in order to build this navigation improvement. Although the rock-filled section was about 56 percent of the total dam's length, the rock-filled length of 1,800 ft (547 m) was constructed to be 210 ft (64 m) wide at the bottom and 30 ft (9 m) wide at the top. Four rock sizes (6 tons to 8 in.) were placed in water depths to 30 ft (9 m) using barge-mounted rock grapple cranes and a specially designed tremie pipe (fall-pipe). Rock was placed in 5-ft (1.5-m) lifts in a sequential fashion described by Smith (1962).

Another hydroelectric dam (Wanapum Dam) built on the Columbia River was completed in 1963. Instead of constructing an impervious bottom blanket as done at Hugh Keenleyside, the river was diverted, a cutoff was constructed, and an embankment dam was completed with an impervious core over a grouted slurry trench. Underwater backfilling of pervious sand and gravel was accomplished between upstream and downstream rock-fill dikes. Underwater consolidation of the pervious fill (maximum depth of 50 ft or 15 m) was performed using vibroflotation equipment. After consolidation was completed,

the slurry trench was constructed, and the remaining dam was constructed as detailed by Engstrom (1963).

Bridges

Underwater foundations have been constructed in the wet for numerous bridge structures crossing rivers, bays, and nearshore locations.

Brooklyn Bridge

It took 13 years to construct this bridge over New York's East River. Work began in 1870 on a unique and labor-intensive method of constructing underwater foundations. Two wooden caissons were sunk on either side of the river upon which the bridge superstructure rests. The unique features included the size of the caissons and the use of compressed air beneath the caissons, which allowed the workers (sandhogs) to muck out the soil. As the soil was removed, the weighted caissons eventually settled to the bedrock layer and were filled with concrete. Compressed air had been used before during construction of the Mississippi River bridge at St. Louis, but not to the extent used at the Brooklyn Bridge.

The caisson on the Brooklyn side of the East River was a 168-ft-long by 102-ft-wide by 15-ft-deep ($51 \times 31 \times 5$ m) inverted box. It was constructed upriver in the dry using wood timbers, and weighed 16 million pounds (7.2 Gg). It was floated downriver, set within a frame, and weighted down with stones until it penetrated the river bottom. Compressed air was pumped in, which expelled the water and allowed entrance by the sandhogs (Figure 7). Many lives were lost before the caisson foundation was excavated down to the bedrock layer, and compression sickness (caisson disease or the "bends") injured many of the sandhogs (Delaney 1983).

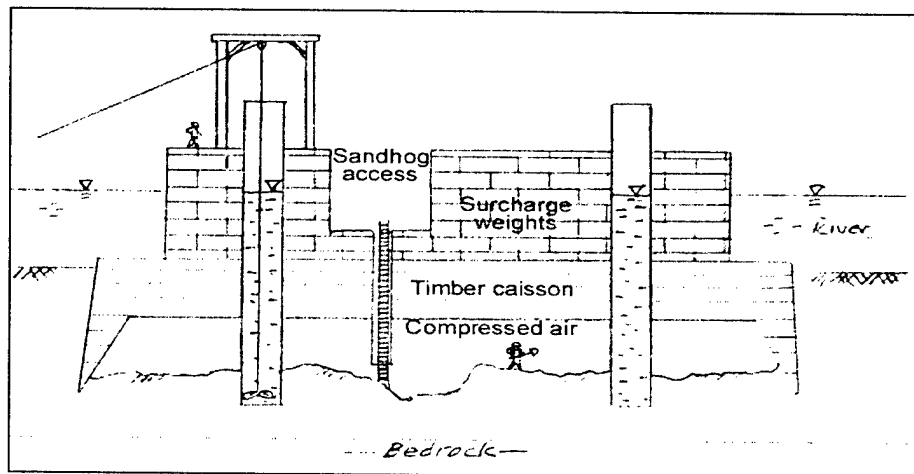


Figure 7. Sketch of Brooklyn Bridge underwater caisson (after Delaney 1983)

Tappan Zee Bridge

The 6-mile-long (9.6 km) Governor Malcolm Wilson Tappan Zee Bridge, which crosses the Hudson River about 13 miles north of New York City, is one of the largest bridges in the United States. The bridge is named from the local Tappan Indian tribe and the word “zee” is Dutch for “sea.” The bridge is the key structure on the 641-mile New York Thruway System and was rededicated in 1994 in honor of the former Governor Malcolm Wilson. Construction began in 1952, and the bridge became operational in 1955. More than 40 million vehicles presently use the bridge each year (New York State Thruway 1999).

The central span over the main channel is supported by eight concrete caissons on steel H-piles driven to bedrock (Figure 8). The concrete caissons are basically hollow concrete boxes that were built on shore, then towed into the channel and sunk onto prepared 5-ft-thick (1.5-m) sand and gravel blankets on the river bottom. Steel sheet piles surround each concrete caisson. The steel H-piles were then driven through the box to depths of 270 ft (82 m) to the bedrock layer. As the upper bridge supports were constructed, the water inside the boxes was pumped out to provide a buoyant platform. Approximately 70 percent of the structure’s dead weight is supported by these eight buoyant caissons, and this design saved millions of dollars during the construction phase.

Newport, RI, bridge

New England’s largest suspension bridge (the Claiborne Pell bridge) crosses Narragansett Bay with a water depth of 160 ft (49 m) and bottom current velocity to 4.5 fps (1.4 m/sec). The \$17 million foundation system was constructed beginning in 1965 and is composed of steel H-piles and drilled shafts supporting tremie-concreted footings resting on backfilled sand (Heddefine and Silano 1968).

Dredging to remove 20 ft (6 m) of silt and unsuitable material was accomplished at each bridge pier. Next, 512 steel piles (14BP102) 70 ft (21 m) long were driven to refusal in shale and dense sand formations at spacings of 4 ft (1.2 m) on center. A special pile-driving lead with retractable spuds at the lower end was built to accommodate piles driven by a McKiernan-Terry S14 hammer. Lateral deviation of 6 in. (15.2 cm) was permitted. Divers in a diving bell made the underwater cutoffs using an electric oxygen arc. After the piles were cut, a sand/gravel fill (40 to 80 percent passing the 3/8-in. sieve with less than 5 percent passing the No. 200 sieve) was placed on the bottom. Template guide pipe piles (casings) were then driven to assist with placement of the bottom footing form.

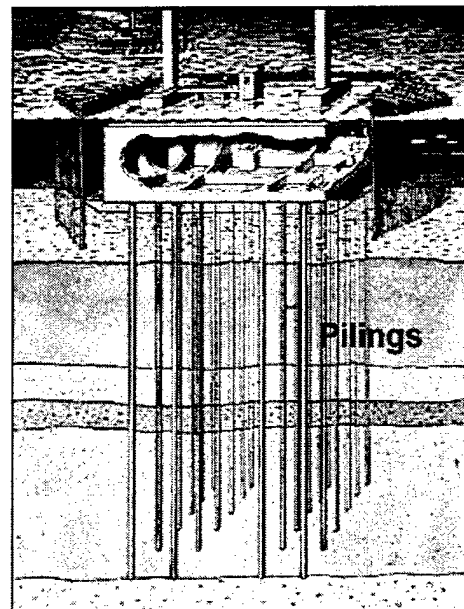


Figure 8. Tappan Zee Bridge foundation (after New York State Thruway 1999)

Each steel footing form, including the attached steel reinforcement bars, weighed over 400 tons and required a heavy-lift crane vessel to sink it onto the pile template. After the footing form was rested on the bottom, a steel hollow-shaft caisson form was lowered onto it. Both forms were tremie-concreted in a continuous operation. Graded riprap (50 to 100 percent passing the 12-in. mesh, 20 to 50 percent passing the 5-in. mesh, and 0 to 20 percent passing the 1.5-in. mesh) was placed around the completed foundation to prevent 4-ft (1.2-m) depth maximum design scouring.

Confederation Bridge

The world's longest continuous crossing over water subject to ice floes was constructed during a 4-year period and spans the Northumberland Strait between Canada's Prince Edward Island and New Brunswick.

The bridge was built on precast concrete piers and base foundations (Figure 9). Each base weighed almost 6,000 tons and was lowered into place on the seabed. Each base underside had three pedestals that were set at different elevations to keep the base elevated at the correct grade on the sloping seabed. To ensure uniform stress transfer between the concrete structure and seabed, concrete was tremied between the base underside and the foundation rock. Tremied concrete was selected instead of compacted gravel because it was believed that better placement accuracy and better settlement resistance would be achieved with concrete (Carter 1998).

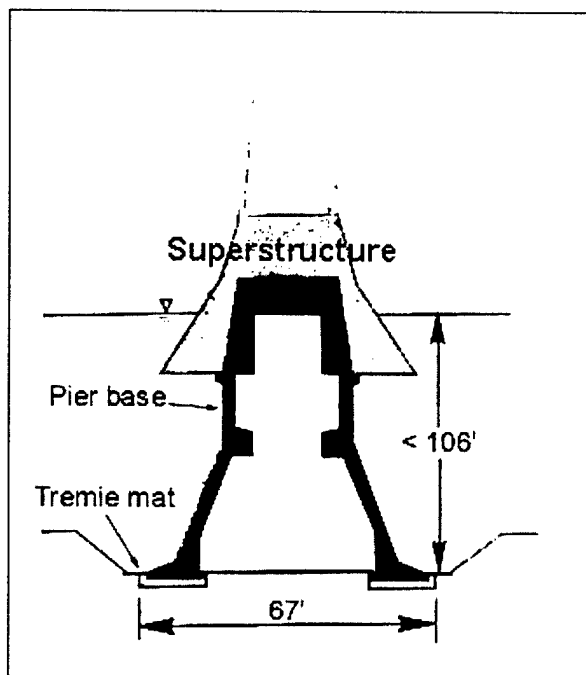


Figure 9. Confederation Bridge foundation (after Carter 1998)

Great Belt eastern bridge

This bridge spanning the shipping channel between the Baltic Sea and the North Sea is the last part of Denmark's Great Belt transportation link, which opened in 1998. Unsuitable material with low bearing capacity was discovered after construction began, requiring additional excavation dredging. Although caisson construction took place offsite, the foundation preparation was done prior to towing and positioning the caissons. Some of the methods used to construct the improved-site foundation included dredging with a bucket dredge to minimize disturbance of boulder clay, clearing sediment siltation with suction equipment, placing crushed stone material in layers with a side-dumping vessel, performing vibration

compaction with powerful plate vibration units, and screeding the foundation top layer level to a 2-in. (50-mm) tolerance (*Gerwick News* 1992).

Akashi Kaikyo Bridge

The world's longest suspension bridge (1.9-km center span) crosses the Akashi Strait in Japan (Figure 10). Completed in 1998, the bridge construction took 10 years. Foundation preparation required dredging to a consolidated gravel layer 46 ft (14 m) below the seabed. Steel caisson foundations were floated in and sunk into position in water currents up to 13 fps (4 m/sec). During placement of the foundations, a wave-breaking apparatus was used to allow accurate positioning and placement of the caissons. High-performance concrete was then placed by tremie pipe (Yao, Berner, and Gerwick 1999). After the caissons were in place, graded aggregate was placed based on hydraulic laboratory scour protection testing results conducted by the Honshu-Shikoku Bridge Authority (1996). In 1995, during construction, the Kobe earthquake epicentered 2.5 miles (4 km) from the bridge and shook the foundations. Although the span distance was increased (less than 3.2 ft (1 m)), it was concluded that the foundations and anchorages had not been damaged (Kajima Corporation 1998).

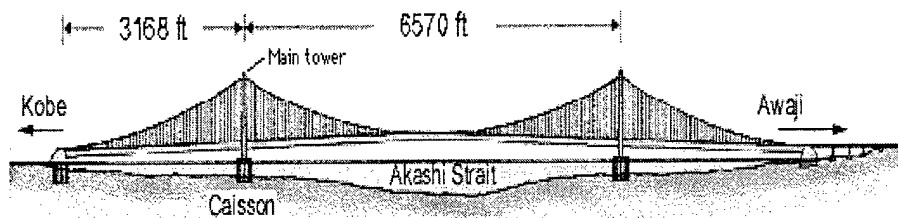


Figure 10. Akashi Bridge (after Kajima Corporation 1998)

New Benicia-Martinez Bridge

A parallel crossing of the Carquinez Strait near San Francisco is currently under construction and will be completed by 2003 (ASCE 1999b). The new bridge (I-680) is designed to withstand a 1,000-year-return earthquake (the maximum credible earthquake) and will be constructed in water currents of 7 fps (2 m/sec). The bridge ductility requirements called for unusually deep drilled shafts into the underlying weathered sandstone bedrock. The bedrock layers are uneven, and dip at angles up to 70 deg. The water depth ranges to 60 ft (18 m), underlain by mud deposits to 85 ft (26 m) deep. Severely weathered bedrock underlies the mud.

During feasibility and design studies, the scouring effects of additional piers next to existing bridge piers were considered. Six types of candidate foundations

were analyzed (*Gerwick News* 1990), and final design selection was 7.2-ft (2.2-m)-diam steel casings drilled 60 to 100 ft (18 to 30 m) into the bedrock. Steel reinforcing will be placed in the shafts and filled with tremied concrete. The rock socket depths were chosen based on the seismic design bending moments and overturning forces.

Pocomoke River Bridge rehabilitation

A bascule pier drawbridge originally built in 1921 over the Pocomoke River in Maryland rests on wood piles that became severely exposed due to riverbed scour. To stabilize and support the bridge, grouted pinpiles were installed (Bruce 1992). Each of the 24 piles was drilled from the bridge deck approximately 60 ft (18 m) through the soft mud bottom into a dense sand layer. Each pile consists of a 7-in. (18-cm)-diam steel casing into which epoxy-coated rebar and wire tendons were placed and grouted (Figure 11). The wire tendons were prestressed

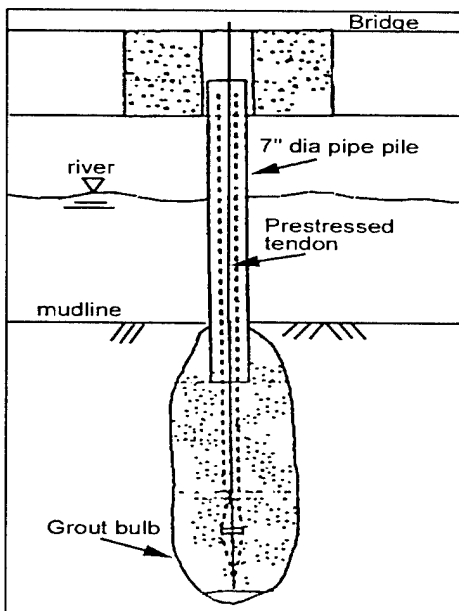


Figure 11. Pocomoke River Bridge in-the-wet foundation rehabilitation with pinpiles (after Bruce 1992)

to 82 kips (365 kN) and released about a week later to allow the design pinpile load of 82 kips to be mobilized without allowing bridge deck settlement. An on-land validation of the design was conducted using test piles driven through an 8-in. (20-cm)-diam outer casing to simulate underwater conditions. After driving, the test piles were loaded to 200 kips (889 kN) after having been prestressed to 82 kips to obtain load/deflection data. Permanent displacement was less than 0.1 in. (0.25 cm).

Immersed Tube Tunnels and Pipelines

The basic concept of the immersed tunnel sunk in an underwater trench dates back to the ancient Babylonians (Palmer and Roberts 1975). Numerous tunnels, sewer lines, pipes, and other subaqueous structures have been constructed using the immersed tunnel concept, where the structure is ballasted and sunk onto a prepared bottom or trench and then may be deballasted after coupling.

Various types of foundations support these structures, including tremied/backfilled sand bases, compacted sand and gravel bases, grout/concrete pads, adjustable jacks, and caissons.

Immersed tube tunnels include the Detroit River tunnel, the Detroit-Windsor tunnel, Mobile's Bankhead and I-10 tunnels, the Texas Baytown tunnel, Virginia's Hampton Roads and Chesapeake Bay tunnels, Baltimore's Fort McHenry tunnel, and the Netherlands' Maas River tunnel (Palmer and Roberts 1975). Others include Tokyo's Dainikoro tunnel (Paulson 1980) and Greece's Aktion-Preveza tunnel (Geotronics 1997).

Bay Area Rapid Transit (BART) Tunnel

Completion of the Trans-Bay Tube segment of the San Francisco BART in 1969 was considered a major civil engineering feat. Constructed in 58 sections and reposing on the bay floor at depths to 135 feet (41 m), the \$180-million project took 6 years to design and 3 years to contract and build (Brugge 1974). Watertight composite steel and concrete sections were fabricated in a dry dock, launched, and towed to the site. Each section was ballasted and sunk into a pre-excavated trench (Figure 12).

The sections were then connected and welded into one continuous structure. Instead of excavating to bedrock, the structure lies on a layer of softer deposits for the purpose of damping possible seismic shocks (Kassel 1986).

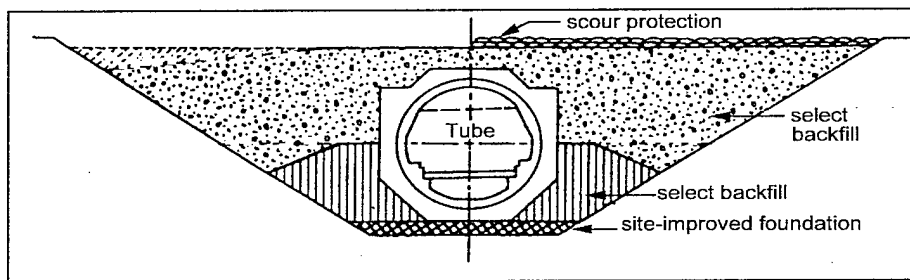


Figure 12. BART Tunnel (after Kassel 1986)

Boston Tunnel

The Boston Central Artery/Tunnel project is expected to cost \$10.8 billion. It will replace elevated highway structures with underground tunnels linking interstate highways and the airport upon final completion in the year 2004. Six concrete immersed tube sections provide the crossing under the Fort Point Channel. Each 48,000-ton tube section was precast on shore, winched and positioned using global positioning system surveying technology, and sunk into a 20-ft (6-m)-deep dredged excavation (Green 1996, Angelo 2000). Unlike other immersed tunnels, each tube section rests on caissons drilled into the subsurface below the dredged excavation (Figure 13). The additional foundation supports were needed to prevent damage to an underlying subway tunnel. Should the immersed tube sections accidentally fill with water, they could cause the underlying tunnel to fail (Brudno and Lancelloti 1992).

Chek Lap Kok Airport transportation links

Hong Kong's new airport construction involved multiple contracts providing transportation linkages, including highways, railroads, and ocean terminals. The final link to Hong Kong's central district and the third underwater crossing of Victoria Harbor is a 1.25-mile (2-km)-long submersed highway tube connecting Hong Kong and West Kowloon. The tube segments were precast on land and

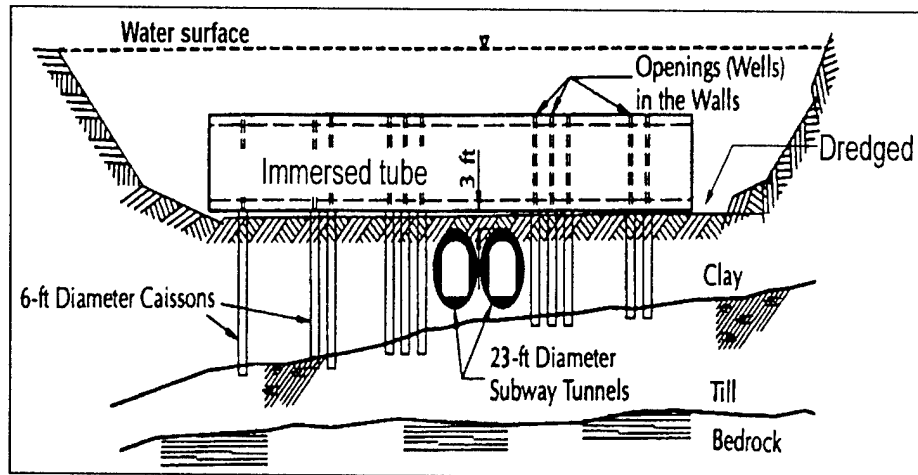


Figure 13. Boston Tunnel foundation (after Brudno and Lancelloti 1992)

towed into the harbor, where they were sunk across its main channel. A trench was dredged across the harbor bottom to receive the tube segments. Horizontal jacks were used to draw the segments together to create a seal. Sand was pumped below the sections to create permanent foundations, and backfilled rock provided protection from scouring erosion and anchor drags. The submerged highway is performing successfully as the final stage of the Airport Core Program (Kosowatz 1995).

Oresund Link

The rail and tunnel connection between Copenhagen, Denmark, and Malmo, Sweden, allows an unprecedented direct link between those two countries. The western end of the link is the world's longest immersed tube tunnel (2.5 miles, or 4 km) carrying both a railroad and highway. It is composed of 20 precast concrete segments (55 kt each) floated into place and positioned onto a prepared gravel base within a dredged trench (Oresundskonsortiet 1999).

Puget Sound gas pipeline

Dual 8-in. (20-cm)-diam pipelines were placed in 600-ft (182-m)-deep water in Washington's Puget Sound in 1969 (Bomba and Seeds 1970). At the time, this was the world's deepest pipeline project. Geotechnical information was gathered using geophysical subbottom profiling. Sediments obtained by core barrel sampling ranged from boulders to silty clays. Although no deep trenching was performed, 4-ft (1.2-m) layered rock placement around and over the pipe was accomplished using a barge-mounted clamshell bucket and a tremie (fall-pipe) hopper. Inspection by divers (to 150-ft depth) and video cameras verified the effectiveness of the placement techniques.

North Sea gas pipeline

The Ekofisk oil and gas production complex in the middle of the North Sea began development in the 1970s. Many “firsts” were accomplished at this site, including the first major offshore gravity-based foundation for a concrete underwater oil storage tank (Focht and Kraft 1977).

Major underwater pipelines were constructed to transport the oil and gas to the shores of Norway, Denmark, England, and Germany. The longest pipeline carries gas to Emden, Germany, for a distance of 275 miles (440 km). This 3-ft (1-m)-diam pipeline is laid to a maximum seabed depth of 230 ft (70 m) in a backfilled trench. Trenching operations to approximately 10 ft (3 m) below the seabed were conducted using a “jet sled” method and were backfilled with natural (sand) material. Minimum cover requirements of 20 in. (50 cm) were regulated to minimize accidental ship anchor and trawler damage to the pipeline. Postconstruction surveys indicated that approximately 10 percent of the pipeline had backfill less than 20 in. (0.5 m), so engineered backfill operations were conducted after the pipeline was operational.

Two backfill systems were designed and implemented. The first was trailing suction hopper dredges that dredged and transported fine sand and gravel, respectively, to a drill ship. The drill ship conveyed the respective materials to the pipeline via a fall-pipe system. The second system was a bulk-carrier ship converted to haul the respective materials to the pipeline site, then convey them to the pipeline using a fall-pipe system with attached electrohydraulic thruster units for precise backfill acoustic positioning over the pipeline. To backfill the approximate length of 23 miles (37 km) with these two systems took about 2 years and cost twice as much as the original pipeline laying and trenching operation (Loeken 1980).

Los Angeles sewage sludge outfall

In 1957, a steel 22-in. (56-cm)-diam pipeline was pulled 7 miles (11.2 km) offshore and sunk in 300-ft (91-m)-deep water within a 7-day period. Specially designed equipment made this feat possible. After the pulling operation was completed, a unique submarine pipeline trencher with jet nozzles straddled the pipe as it laid on the seabed and trenched about 6,000 ft (1,824 m) from the shoreline into the ocean. As high-pressure water and air jetted through the nozzles, the bottom material was cut away, allowing the pipe to settle into the trench. The deepest trenching (about 15 ft or 5 m deep) was performed in the surf zone, where sheet piles were used to protect the trench excavation. Other unique equipment and installation techniques are discussed in *Construction Methods and Equipment* (1957) and Narver and Graham (1958).

Puerto Rico sewage outfall

The world’s deepest ocean sewage outfall was recently constructed off the coast of Ponce. The 3.5-mile (5.6 km)-long, 4-ft (1.2-m) pipeline lies at a

maximum depth of 450 ft (137 m) on the seafloor. The pipe sections were towed into position and lowered from barges to the seabed. Deep anchors consist of grout-filled bags lowered from the surface. Shallow mud deposits required an excavated trench into which the pipe was placed to achieve neutral buoyancy (Powers 1997).

Offshore Oil Platform Structures

Perhaps the best examples of underwater foundation in-the-wet construction are found in the offshore industry's structures, due to their unparalleled sizes, costs, complexity, and remote locations around the world. Offshore (outside sheltered waters) oil exploration and production began after World War II off the Louisiana coast. The first offshore steel-piled platform was fabricated in Morgan City, LA. Seven years later, the first jack-up drilling rig was placed offshore. Since that time, more than 5,500 platforms have been installed in the Gulf of Mexico (Hunt and Gary 2000).

Water depths of 20 to 50 ft (6 to 15 m) were typical for the first fixed platforms erected on steel pipe pile foundations. The 1- to 2-ft (0.3- to 0.6-m)-diam piles were typically driven to refusal to support loads of 200 to 400 kips (890 to 1,780 kN). Gradually, the fixed platforms were built in water depths to 200 ft (61 m), and significant advances were being made in marine geotechnical engineering.

Proprietary research programs were being conducted by the oil companies for the purpose of reducing design uncertainties since the test and performance databases were incomplete. For instance, land-based pile load tests ranged to only 300 tons (1,320,000 kg), whereas the typical offshore pile load was about 2,000 tons (8,800,000 kg) (Focht and Kraft 1977).

Progress was also made in other aspects of marine geotechnical engineering, including tension-loaded piles, laterally loaded piles, earthquake and hurricane hazards, and new types of foundations (e.g., suction caissons). Mobile drilling platforms consisting of pile-and-mat supported jack-up platforms and tension-leg platforms (TLP) supported by tension pile foundations were being built in the Gulf of Mexico. In the North Sea, the typically encountered clay seabed dictated that other foundation systems be designed, including drilled pile foundations and gravity-based foundations. Technology for gravity-based structures (GBS) has since been applied in the Gulf of Mexico (Huntman, Anastasio, and Deshazer 1979). Suction caissons were first installed in the North Sea (Senpere and Auvergne 1982), and they have since been used in the Gulf of Mexico. Adaptations of the suction caisson technology have been applied to jacket platforms, jack-up platforms, TLPs, and "skirted" GBS foundations.

In general, offshore structures are attached to the seabed either rigidly (fixed structures) or flexibly (floating structures). Those rigidly attached to the seabed may be either fixed or mobile. Structures that are not attached to the seabed are mobile and may be production platforms, storage facilities, off-loading facilities, or exploratory drill platforms. The mobile nonattached structures are not

discussed here since they are dynamically positioned and do not require foundation preparations. Most oil production platforms are attached to the seabed either rigidly (jackets, jack-ups, gravity-based, and compliant structures) or flexibly (floating TLP, mini-TLP, and spar structures). The foundation systems for the platforms are either pinned (driven, drilled, or suction-induced) or gravity-based, or combinations of the two.

Jacket and jack-up structures

Offshore oil production platforms are commonly built on a fixed “jacket” structure that is a tubular steel frame that is either floated or lifted in, positioned on the seabed, and pinned with pilings (Figure 14).

A “jack-up” structure is intended to be mobile and has a modified foundation including a “spud can” arrangement that is stabbed into the seabed floor (Figure 15).

The first mobile offshore platform designed for use in 100-ft (30-m) depths with a soft soil foundation was a pile-and-mat supported jack-up platform (Focht and Kraft 1977). It consisted of six 4-ft (1.3-m)-diam pipe piles supporting an 80-by 100-ft (24-by 30-m) steel mat (Figure 16). The bearing pressure on the mat base ranged between 200 and 400 psf (9.6 to 19.2 kPa) depending on the deck load. This foundation system is a combination gravity base underpinned by driven piles and was installed in the Gulf of Mexico.

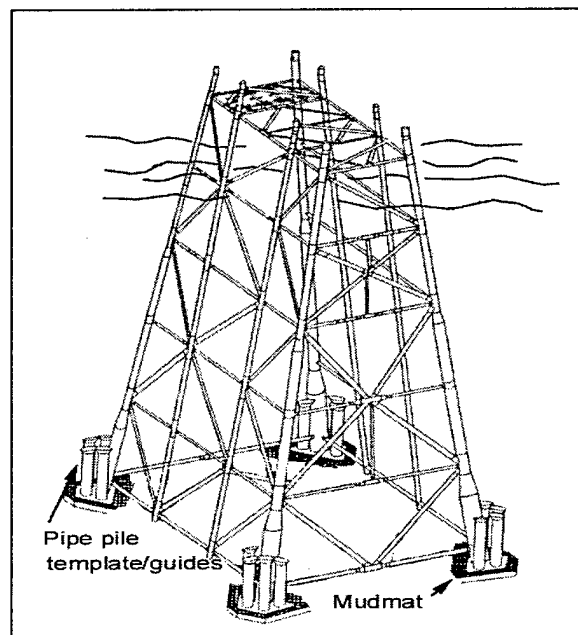


Figure 14. Fixed jacket structure (after White and Drake 1994)

Most jacket platforms have pinned foundations with driven piles, and are the most common type of offshore oil platform (Lacasse 1999). They are located worldwide, including the Gulf of Mexico, North Sea, Atlantic, and Pacific Oceans. For example, Exxon decided to use a jacket platform for its Hondo structure in California’s Santa Barbara Channel. The Hondo platform’s steel pipe piles (54-in. diam \times 375-ft length) were driven using nonsubmersible Vulcan and Menck hammers above the 850-ft (258-m)-deep ocean floor (Bardgette and Irick 1977).

The Alba Northern jacket structure installed in the North Sea has four steel mudmats to which pipe pile templates are attached. The single-launched jacket was floated in and installed in 414-ft (138-m) water. Each footing was pinned by

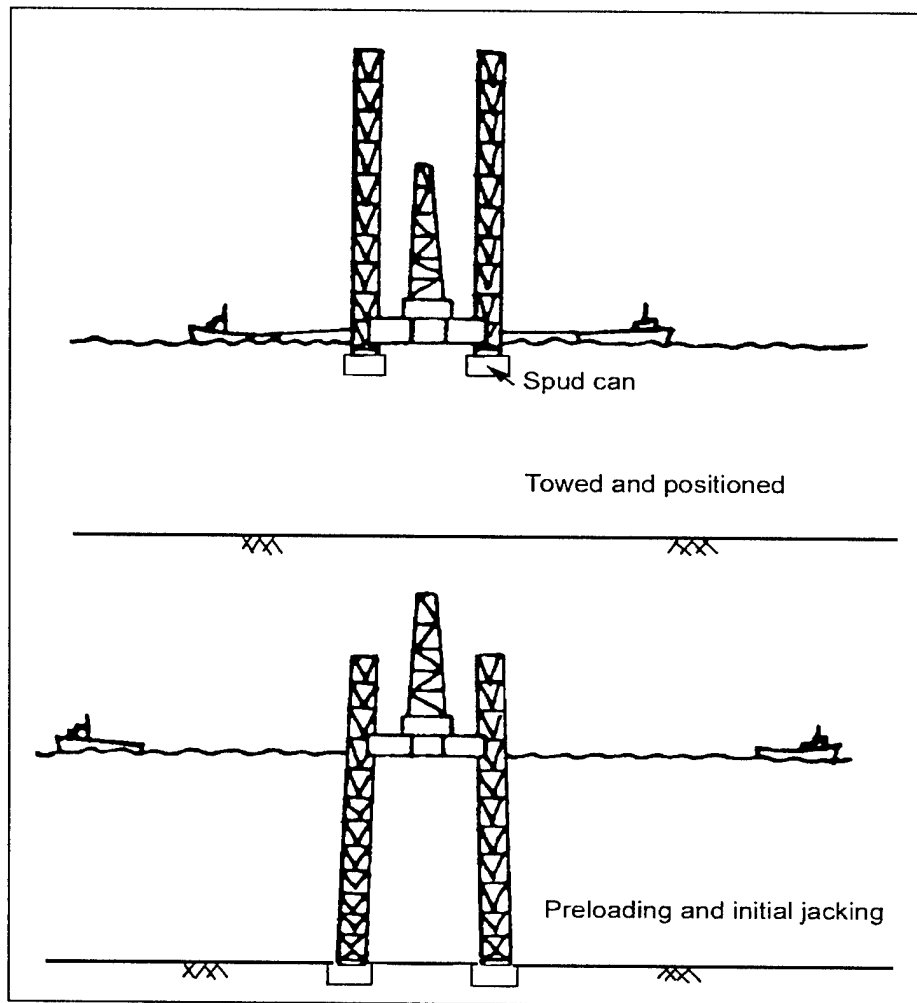


Figure 15. Jack-up structure installation (after Kobus and Whittington 1978)

four 96-in. (2.4-m)-diam piles that were driven 264 ft (88 m) with hydraulic-powered underwater hammers (White and Drake 1994). An innovative antifouling coating of cupro-nickel sheeting bonded to neoprene was applied to the steel structure to prevent marine growth.

An alternative to single-launched jacket designs for water depths in the 500- to 700-ft (167- to 233-m) range is Shell's two-piece grouted Enchilada platform in the Gulf of Mexico (Dorgant et al. 1998). After setting the base section, eight 84-in. (2.33-m)-diam steel pipe piles were driven to 385 ft (128 m) using the free-riding IHC S-400 underwater hydraulic hammer without any supporting guides. The jacket top section was lifted into position and grout-sealed.

Shell's Cognac jacket platform was installed 1,025 ft (342 m) below the Gulf of Mexico surface and was the world's first three-segment jacket (Sterling et al. 1979). The pinned foundation consists of 24 pipe piles, 7 ft (2.3 m) in diameter and 625 ft (208 m) long, driven through outrigger-type skirt pile sleeves. Each pile penetrated 450 ft (150 m) into the seabed.

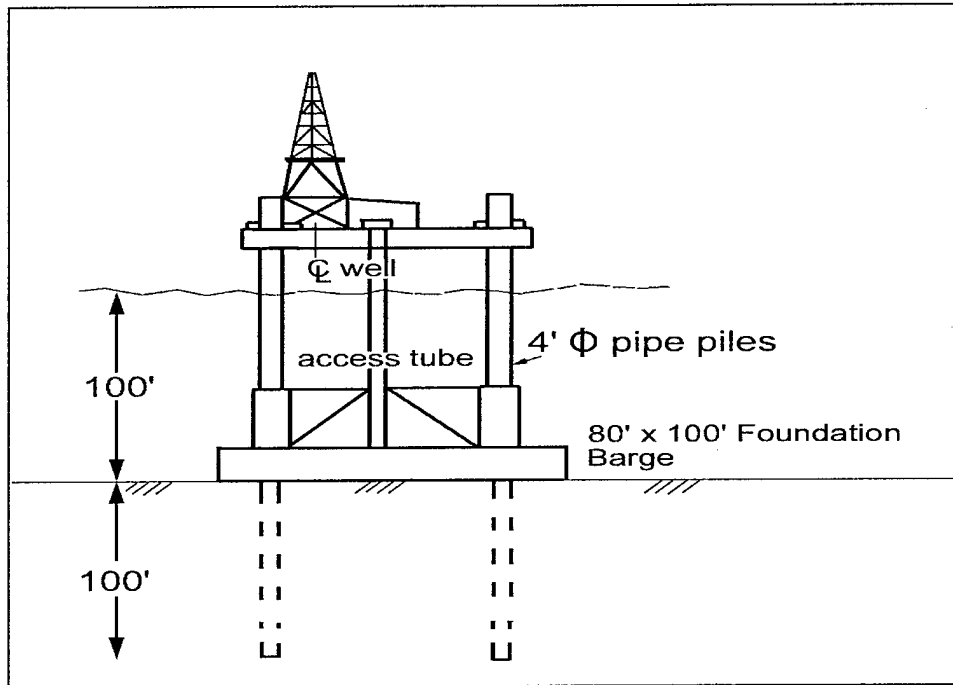


Figure 16. Jack-up platform on soft soil (after Focht and Kraft 1977)

The world's largest installed jacket structure is the Bullwinkle, at a Gulf of Mexico water depth of 1,350 ft (400 m) (Offshore Technology 1999).

Combining driven piles with drilled insert piles was required for the construction of the Goodwyn "A" steel jacket platform off Australia. Calcareous soil caused difficulty driving 8.7-ft (2.65-m)-diam piles past 382 ft (116 m) below the mudline. Therefore, 6.5-ft (2-m)-diam insert piles were drilled to a depth of 595 ft (181 m) below the mudline. The annulus between the piles was grouted with a specially designed grout mix (*Gerwick News* 1993).

Suction-pinned foundations replaced traditional piled foundations on Statoil's innovative Europipe jacket platform in the North Sea. The circular ("bucket") foundations with skirts increase the vertical compression and shear load capacity and provide substantial tensile load capacity (Baerheim, Hoberg, and Tjelta 1995).

The Europipe four-leg jacket (Figure 17) stands in 210 ft (70 m) water. Each bucket has a diameter of 36 ft (12 m), weighs 192 tons (215 tonnes), and penetrates about 18 ft (6 m) below the seabed. The jacket was lifted in, positioned on the seabed, and the buckets were suction-pumped to achieve penetration. The void between the seabed and the bucket top was then grouted to ensure an even support.

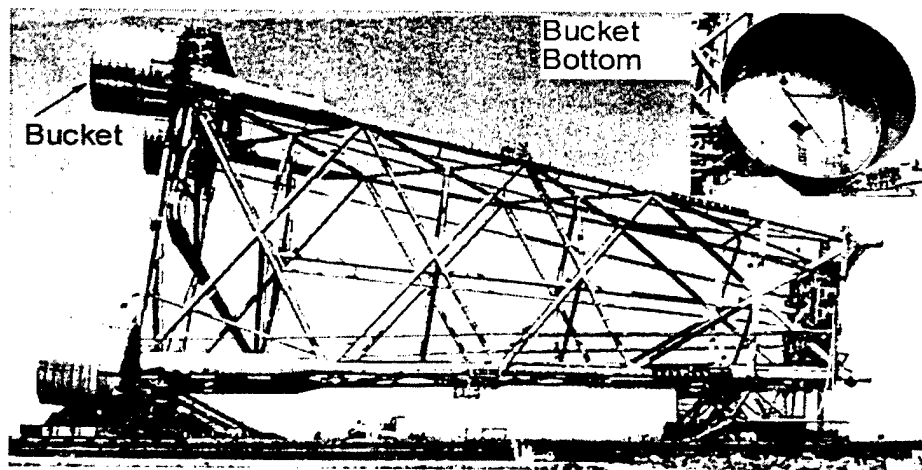


Figure 17. North Sea fixed jacket with bucket foundation (after Baerheim, Hoberg, and Tjelta 1995)

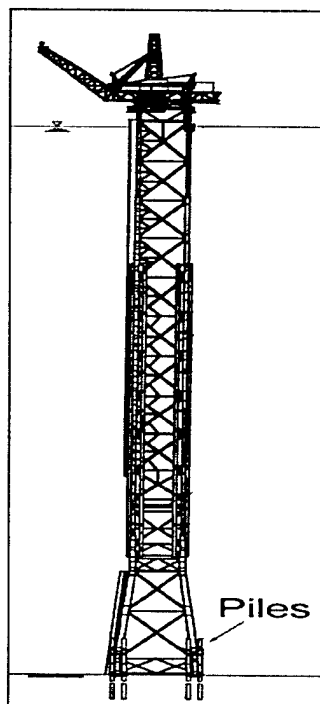


Figure 18. Compliant tower platform (after Will 1999)

Compliant tower structures

The compliant tower (CT) is a modified steel jacket structure (Figure 18) that occupies a much smaller footprint on the seabed (Figure 19). It is designed for sway periods of about 30 sec instead of typical platform sway periods of about 3 sec. As a result, resonance is reduced and wave forces do not amplify the natural vibration frequency of the structure. Maximum design displacement (100-year hurricane event) is 1.5 to 2 percent of the water depth. In contrast, floating systems (TLP and Spars) may have lateral movements of up to 10 percent of the water depth.

The idea of the CT concept was initially used in the Bay of Biscay in 300-ft (91-m) water depth (Appert and Burger 1997). The most recent deepest CT installations are Amerada Hess' Baldplate installed off Texas in 1,650-ft (550-m) water depth and Texaco's Petronius installed in 1,754-ft (583-m) depth in the Gulf of Mexico, although Petronius' production wells are not scheduled to be on-line until 2001 (Will 1999).

The Baldplate is the first freestanding, nonguyed CT and was the tallest free-standing structure in the world when constructed. Its 12 pipe piles (diameter 7 ft (2.1 m); length 428 ft (130 m)) each have an ultimate capacity of 8,400 tons (74.7 MN) (Will et al. 1999).

The TLP foundation resists tensile and lateral forces imposed by semi-submersible structures (Figure 20). Many design variations have been used for offshore platforms, but the principle is the same (providing an anchoring system for floating platforms).

Tension-leg platform structures

Conoco's North Sea Hutton TLP was installed in 1984 and is the world's first TLP oil production platform (Stock, Jardine, and McIntosh 1993). The site, 475 ft (145 m) below water surface, is underlain by stiff glaciomarine tills and dense sands. Each of the four pinned foundations has eight 6-ft (1.8-m)-diam pipe piles driven 200 ft (60 m) below the mudline, and each pile group normally resists a tensile load of 1,100 tons (1,000 metric tons) and a surface platform lateral offset of 66 ft (20 m).

The largest TLP in the Gulf of Mexico (and the deepest TLP in the world) is Conoco's Ursa located in 3,800-ft (1,267-m)-deep water (Digre, Kipp, and Hunt 1999). Sixteen 96-in. (2.4-m)-diam pipe piles, 417 ft (139 m) long, provide the pinned foundation. Each pile self-penetrated 125 ft (42 m) into the seabed prior to underwater driving.

The majority of TLP foundations are pinned with driven piles, but gravity-based foundations are also used. For example, Conoco's North Sea Heidrun TLP is tethered to four concrete gravity-based foundations (Mitsha, Morrison, and Oliveira 1997).

For "marginal" oil production sites, more economical "mini-TLP" platforms are constructed, which are typically smaller and faster to install. British Borneo's Morpeth platform (Figure 21) is a mini-TLP installed to a depth of 1,670 ft (557 m) in the Gulf of Mexico. Six pipe piles, each 84 in. (2.3 m) in diameter by 340 ft (113 m) long were driven into the seabed. Each 214-ton (195,000-kg) pile self-penetrated to 130 ft (43 m) prior to driving (Redfern, Calkins, and Matten 1999).

Other variations of the TLP concept include the Spar and Deep Draft Caisson Vessel (DDCV). Examples of each are the Gulf of Mexico's Neptune production Spar platform and the Hoover Diana DDCV (Offshore Technology 1999).

The Snorre platform in the North Sea became the first TLP to use suction caissons instead of driven piles (Figure 22). Concrete gravity cells with perimeter skirts (suction caissons) of 66-ft (20-m)-diam were installed in very soft clays at a water depth of 1,050 ft (320 m). Piles were determined to be unacceptable because of extremely low clay soil friction (bearing) capacity.

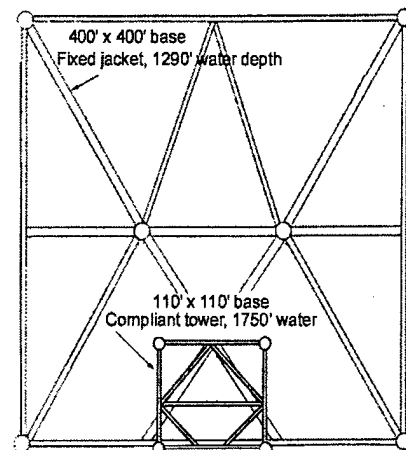


Figure 19. Compliant tower foundation footprint (after Will 1999)

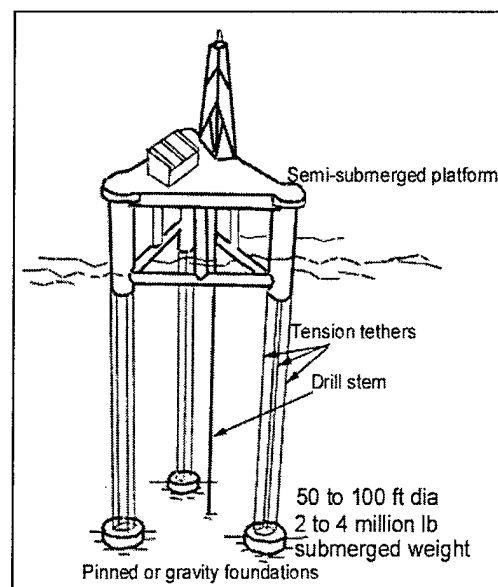


Figure 20. Typical TLP

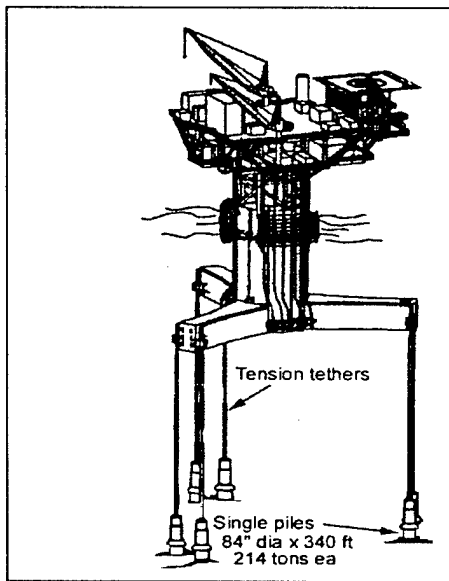


Figure 21. Mini-TLP structure (after Redfern, Calkins, and Matten 1999)

Each cell group (out of four total groups) has a total submerged weight of 6,250 tons (7,000 metric tons) after being ballasted with 3,125 tons (3,500 metric tons) of iron ore and olivine. The 100-year design storm maximum tension load for each cell group is 12,678 tons (14,200 metric tons), and the average tension load is balanced with the cell group weight. Accounting for the cell group weight, the average long-term load experienced by the soft clay soil is zero (Christophersen 1993).

Gravity-based structures

The advantage of gravity-based over pinned structures is their shorter installation duration, which is a big factor in reducing exposure to hostile storms in the North Sea and North Atlantic. The structures are constructed onshore or in protected waters, towed into position, and sunk to the sea floor. Installation can be accomplished during good weather windows (Young, Kraft, and Focht 1975).

The first major offshore gravity foundation was placed in the North Sea in 1973 (Focht and Kraft 1977). It was a 305-ft (93-m)-diam concrete oil storage tank in the Ekofisk Complex weighing 490,000 tons (444 million kg) that was a float-in structure installed in 230-ft (70-m) water depth. No piles were used to support the structure.

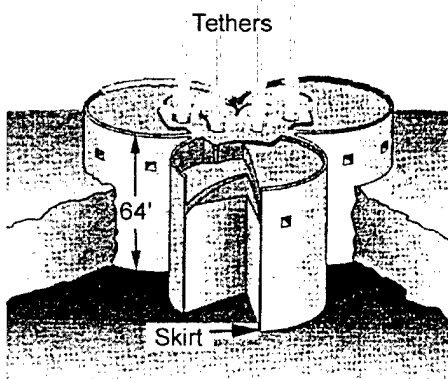


Figure 22. TLP suction pile foundation (after Christophersen 1993)

Another offshore foundation “first” occurred on another Ekofisk Complex structure. Drilled and underreamed shafts were constructed in the hard clay seabed instead of driven piles. The land-based practice of installing drilled shafts instead of driven piles at stiff or hard clay sites was common, but had not been used offshore, especially in water depths to 350 ft (107 m) (Focht and Kraft 1977).

In 1978 the first concrete gravity production platform was installed in the Gulf of Mexico (Huntman, Anastasio, and Deshazer 1979). Although it was actually a combination gravity and pinned foundation structure, it holds a place in offshore foundation history. The offshore site was

located in only 12-ft (4-m) water depth. Foundation preparation of the seabed was required to prevent scour. The seabed was dredged 7 ft (2.3 m) deep, and a 2-ft (0.7-m)-thick oyster shell mat was placed in the excavation. The platform was towed to the site, ballasted, sunk onto the prepared foundation, and pinned with perimeter piles. Limestone rock riprap was then installed around the perimeter to prevent scour.

At the time of its installation, the North Sea's Draugen GBS was the tallest concrete structure ever constructed (Alm, Bye, and Egeland 1995). It is located in approximately 750 ft (250 m) of water on a very hard clay seabed with embedded boulders. Although the upper material had a fairly low undrained shear strength of 312 psf (15 kPa), it increased to 20,800 psf (1,000 kPa) at 18 ft (6 m) of depth. Concrete skirts at the base of the structure penetrated an average of 24 ft (8 m) into the seabed as a maximum suction pressure of approximately 51 psi (350 kPa) was applied.

The North Sea Troll I GBS has skirts that penetrate 108 ft (36 m) into the stiff clay with undrained shear strength of approximately 5.2 tsf (500 kPa). The platform rests in 1,000-ft (303-m) water depth (Andenaes, Skomedal, and Lindseth 1996).

The first offshore gravity-based platform to use suction caissons was Statoil's Gullfaks C (Figure 23) in the North Sea, installed in 1989 (Tjelta 1992). This \$2-billion GBS platform was placed on normally consolidated soft clay at a water depth of 720 ft (220 m). A soil drain system was designed and preinstalled into the caissons. This drainage system allowed faster consolidation of the foundation soil, which resulted in increased strength of the material. Another GBS example is the Hibernia structure installed in the Grand Banks off Newfoundland in 1990 (Offshore Technology 1999).

It is a 495,000-ton (450,000-metric ton) concrete caisson 321 ft (106 m) tall with a 27-ft (108-m)-diam base (Figure 24) ballasted with 452,000 tons (411,000 metric tons) of iron ore (magnetite) and placed in 242-ft (80-m) water depth. The seabed was prepared by "sweeping" with a deep-sea hopper dredger. The caisson has 16 "fingers" to resist icebergs, and was designed to withstand a 500-year event collision with a 1 million metric ton iceberg and a 10,000-year event collision with a 6.6 million ton (6 million metric ton) iceberg.

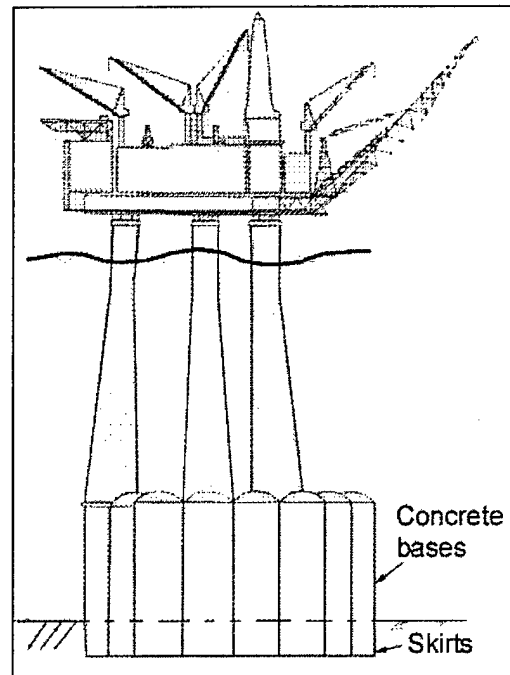


Figure 23. Statoil's Gullfaks C platform (after Tjelta 1992)

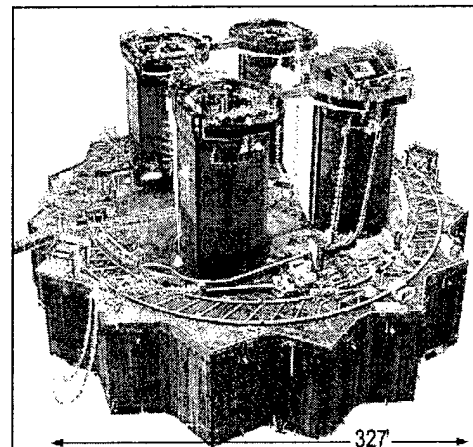


Figure 24. Hibernia gravity-based foundation (after Offshore Technology 1999)

Other Underwater-Founded Structures

Wharf off-loading system

A large-capacity (2,000-ton) transport barge was designed for use at a marine fabrication industrial yard to off-load and deliver equipment at two existing wharfs in Connecticut (Abbott and George 1989). The barge was designed to jack up to the wharf elevation, thus eliminating the need for heavy lifting cranes on the wharf. For each separate wharf, a special underwater foundation was constructed to prevent the spuds from punching through the soft clay (1,000-psf or 49-kPa shear strength), 35 ft (10.7 m) below the water surface. Since the 1,200-ton (10,676-kN) leg design could not be supported by the soft clay, and due to the site conditions at each wharf, two separate foundations were designed and constructed.

At one wharf, bedrock was located 35 ft (10.7 m) below the mudline. Prefabricated steel cofferdam cylinders (diameter 28 ft or 8.5 m) were driven 75 ft (23 m) into the bedrock, excavated, and filled with crushed stone. Precast concrete pads, 20 ft (6.1 m) in diameter and 5 ft (1.5 m) thick, were placed on top of the crushed stone cofferdams to support the barge's spud legs. Placement tolerance of 9 in. (230 mm) for caisson installation was met.

At the other wharf, bedrock was located 50 ft (15.2 m) below the mudline, so other foundation alternatives were considered. The design load for each leg foundation was 2,000 tons (17,794 kN). A reinforced earth pad overlain by a precast concrete base was installed with minimal dredging. Three 10-ft (3-m)-thick compacted pads of crushed stone, sandwiched between mats of steel strips, were placed in a 15-ft (4.6-m)-deep excavated hole. Special procedures were devised to construct the underwater reinforced earth pad. Precast concrete pads, 25 ft (7.6 m) in diameter and 5 ft (1.5 m) thick, were placed on top to support the barge's spud legs (Abbott and George 1989).

Ship dry docks

During the World War II era, the U.S. Navy undertook a massive construction effort to build dry docks for shipbuilding and repair at its numerous naval facilities. Construction took place at Pearl Harbor, Norfolk, Philadelphia, Mare Island, Brooklyn, San Diego, and other locations. Dry docks are classified as graving (basin excluding water) or floating (buoyancy principle). In general, one of two construction methods was used, with the basic difference being the concrete placement method (above-water or tremied). These methods were

- a. Underwater excavation and tremied concrete construction, followed by dewatering.
- b. Underwater excavation concurrently with deep well-point dewatering prior to concrete placement.

Sheet pile installation and dredging operations typically began the construction effort, followed by underwater H-pile installation. The piles were designed to resist uplift during reduction of hydrostatic pressure as the dock was dewatered, and also to carry dock operation compressive loads through any underlying soft layers into a firm stratum below. Construction methods, especially for tremie concrete, were discussed by Harris (1942) and Johnson, Compton, and Ling (1972).

Tate (1961) detailed construction of a Puget Sound floating dry dock with dimensions of 1,180 by 180 ft (359 by 55 m) built to overhaul Pacific Fleet aircraft carriers. Design, construction, and subsequent operation depended on an elaborate network of continuously operating dewatering pumps. After dredging to 40 ft (12 m), the underwater site was backfilled (hydraulic jetting from deck scows) with a sand/gravel mix consisting of 3-in. maximum size with less than 10 percent passing the 100-mesh screen. Sheet-pile bulkheads were installed, the site was dewatered, and in-the-dry construction proceeded. Prior to concrete placement, vibroflotation equipment was used to increase the foundation's bearing capacity (by increasing its relative density).

Millard and Hassani (1971) detailed the construction of Bethlehem Steel's graving dry dock for shipbuilding near Baltimore. The dimensions of the dock were 1,200 by 200 ft (365 by 61 m). This dry dock depended on continuously operated dewatering pumps during shipbuilding operations to reduce hydrostatic pressure against the relatively thin walls and floors. During construction, up to 70 ft (21 m) of soft materials was dredged to expose underlying sand with adequate bearing capacity. Sand was also used as backfill and was placed by bottom-dumping scows (barges). An underwater embankment was also needed as a construction dike. After the site was enclosed by dikes and sheet-pile cells and dewatered, the sand fill was compacted using vibratory pile-driving equipment.

Berlin's Potsdamer Platz

Beginning in 1995, construction began in Germany's new capital to support the government's office move from Bonn to Berlin. The high-rise buildings have basements to depths of 56 ft (17 m) below ground. Artificial lakes and city regulations curtailed dewatering schemes during construction, which required foundations to be built underwater. Pontoon-mounted excavators dug the foundation pits, and divers manually cleaned the bases and positioned sheet piles. Steel pilings were vibrated into the subsurface to resist buoyancy of the concrete base slabs. The base slabs (up to 5 ft (1.5 m) thick) were placed underwater. After dewatering the sheet-pile enclosure, the structural slab was cast in place, and the structure was constructed in the dry (Reina 1996).

Baltic Sea wind turbines

The first offshore pilot projects demonstrating the electrical-generating potential of wind turbines were constructed at locations off the Denmark and Sweden coasts in the Baltic Sea (Danish Wind Turbine Manufacturers

Association 1998). Studies have been conducted not only for the electrical aspects of wind power, but also for the types and costs of underwater foundations.

Four types of wind turbine foundations have been constructed in the wet: concrete (float-in), combination concrete gravity footing with a steel pedestal, steel pile, and steel tripod (jacket) with 3-ft (1-m)-diam steel pile anchors. Installation costs as a function of water depths for each type of foundation were analyzed. In water depths to about 30 ft (9 m), the concrete gravity platforms were the most expensive. The simplest foundation is the single steel pile with diameter between 11 and 14 ft (3.5 and 4.5 m), driven 30 to 60 ft (20 to 30 m) into the seabed. The jacket was the least expensive, but was found to be unsuitable for depths less than 18 ft (6 m) due to tripod interference with approaching service vessels.

Offshore military tracking platforms

In 1978, the U.S. Navy contracted design and construction of four aircraft-tracking tower structures to be built in the Atlantic Ocean 30 miles (48 km) off North Carolina in water depths to 105 ft (32 m). Each tripod tower is supported by underwater 42-in. (107-cm)-diam steel pipe piles driven through a template with penetrations to 270 ft (82 m) below the mudline into dense sands and stiff clays. Rigid design requirements (20-year operational life, excursion limitations of 1 ft lateral and 1 deg rotational in 60-mph winds and sea state 7) dictated applied pile loads up to 1,005 tons in tension and 1,479 tons in compression. The design penetration depths were based on a safety factor of 1.5. Several above-water Vulcan diesel hammers were used to achieve the design penetrations without jetting or predrilling. As an interesting note, predriving analysis using the wave equation predicted that design penetrations could not be achieved (Ling 1978).

Bantry Bay tanker terminal

This deepwater crude oil terminal is located 1,100 ft (334 m) off Ireland's southwestern coast and is founded on battered 40-in. (102-cm)-diam steel pipe piles in 100-ft (30-m) water depth. The piles are 220 ft (67 m) long and were driven through boulder clays and silts to bedrock. A 20-ft (6-m) portion of the tips was rock-socketed by drilling. Initial driving efforts were difficult due to the waves and lack of any driving template. Driven piles left unbraced due to the short winter work schedule envelope failed at weld connections due to the wave action. Larger work platforms, bigger pile hammers, and spud-mounted pile templates corrected this situation. Boulder inclusions in the drilled sockets were reduced by increasing the bit diameter and using powerful airlift ejectors. Reinforcing bars and concrete were then placed in each pile (Fox 1970).

Submerged oil storage tank

The Khazzan Dubai I is a 500,000-barrel oil steel storage tank located 60 miles (96 km) off Dubai in 154-ft (47-m) water depth (Chamberlin 1970). This unique structure has no enclosed bottom (it holds oil by water displacement), and its circular sides are connected to the seafloor by 3-ft (1-m)-diam pipe piles. The structure was towed to its site, ballasted by air displacement, and sunk onto a level (1:100) unimproved seafloor. For each pile, a 42-in. (107-cm)-diam hole was rotary-drilled 100 ft (30 m) into the layered clay, silt, sand, and weathered limestone subsurface, using a tricone bit and underreamer. The pile was lowered into the hole and cemented to both the formation and the structure with expansion grout having 28-day compressive strength of 3,000 psi (21 MPa). Full-scale tests of grouted structure connections were conducted.

3 Foundation Site Assessment

Site and Foundation Selection Process

Site investigations are an essential part of the underwater site selection, foundation design, and foundation installation elements. Adequately planning and executing a site investigation is especially critical for determining the geotechnical influences prior to foundation selection and design (American Petroleum Institute 1989; James 1987; Hitchings, Bradshaw, and Labiosa 1976). Preliminary design efforts require that site surveying and assessment efforts be under way, and the completed site characterization should be finalized prior to final design. Special consideration must be given to the possibility of construction contract problems if a complete site characterization is not conducted prior to final design.

The site conditions (including topography, bottom current, sediment loading and scour, and other environmental conditions) and engineering properties (including soil/rock lithology and stratigraphy, strength, durability, compressibility, and seismic concerns) must be explored and characterized prior to foundation final selection and design (Herrmann, Raecke, and Albertsen 1972).

Topics that need to be addressed during the general site and foundation selection process are outlined below.

Site assessment. Geologic, topographic, and geographic influences are important within the given project's political boundary and should be studied extensively during the site selection process. Nearby industrial or urban situations may impose unique restrictions. In addition to the surrounding land assessment, the seabed/riverbed physical qualities (such as depth, slope, and underwater obstructions) are important. Underwater obstructions include deadfalls, snags, cable crossings, and other sunken objects. The water body's physical, chemical, and environmental characteristics are also vitally important and include the following:

- Tidal variation
- Current profile and hydraulic characteristics
- Hydrographic data

- Icing traits
- Seasonal underwater visibility
- Sedimentation and scouring potential
- Salinity and/or temperature variations, and water chemistry
- Environmental quality considerations and regulations
- Aquatic characteristics (expected problematic species)

Soil and rock engineering properties. A subsurface investigation is vitally important in the foundation selection and design process since it helps the designer understand the soil/rock response to the imposed structural loads. An adequate evaluation of the engineering properties of the seabed or river bottom requires a time-consuming and often costly site investigation, including in situ testing and sampling, laboratory testing, and data analysis. The adequacy of this program may be difficult to assess, given the uncertainties involved for an investigation to be conducted for a remote underwater site. The presence of the water barrier above the site demands innovative or modified utilization of land-based techniques and equipment to achieve reliable and accurate results.

Structure function. The structure's intended function must be properly identified. Two aspects of the structure's foundation function that must be addressed are its intended design life and its degree of confidence requirements (factors of safety, risk analysis, and design uncertainty).

Structure characteristics. Physical size, configuration, submerged weight, load distribution, installation sequence, and dynamic load resistance are important variables to be considered in the foundation selection process. For instance, both land- and water-based structures may be designed to resist wind loads, but the water-based structure must also resist hydrodynamic current drag and wave and ice loads that ultimately are fed into the foundation design requirements.

Installation requirements. Inherent in the foundation selection and design process are the questions of installation equipment and labor, procedures, and quality control. Composition, size, and weight of the foundation components dictate the equipment and methods for transporting, positioning, and installing on the seabed or river bottom. Coordination with above-foundation structural elements' installation requirements will guide foundation selection. Maintaining quality control and verifying quality assurance are important considerations that have to be addressed during the foundation selection process since they will be matched to the specified foundation system.

Contract cost. Balancing the requirement for a reasonable degree of confidence in the foundation's utility with the goal of achieving the lowest project cost is another important variable in the foundation selection process. Reduced competition due to a smaller number of foundation contractors with experience in a given foundation system is not a cost deterrent. As foundation contractors gain more experience and the number of competitors increase, lower bids will result. Consideration must be given to the potential contractors' and subcontractors' capabilities and experience records when selecting a foundation type, not only to

obtain a higher probability for successful installation, but to potentially lower the contract bid amounts.

Site Assessment and Characterization

Noninvasive assessment/characterization techniques, including bathymetric and hydrographic surveys, are conducted to determine the depth and bottom topography. Visual site assessments may be accomplished using remotely operated vehicles outfitted with video cameras. Geophysical surveys such as side-scan sonar are conducted to determine subbottom profiles of debris, sediments, soil, and rock.

Preliminary assessment

Preliminary information-gathering consists of building a site-specific database detailing site location and configuration, water conditions (water depth, density and salinity profiles, temperature variations, current profile from the water surface to the foundation bed, tidal variations), and sediment conditions.

Marine charts typically show navigation aids, water depths, and navigation obstructions including underwater cables and pipelines. Marine charts in digital format are available from the U.S. Department of Commerce's National Oceanic and Atmospheric Administration (NOAA) for coastal areas and the Great Lakes; the Department of Defense's National Imagery and Mapping Agency (NIMA) for open ocean and foreign waters; the U.S. Army Corps of Engineers for inland rivers, lakes, and canal systems; and the U.S. Geological Survey for land-based topography. Georeferenced digital maps with aerial and satellite photography are available from various commercial vendors. The NIMA has recently established a vector-based digital maritime chart database named the Digital Nautical Chart, available freely to U.S. Government agencies with data-sharing agreements and by commercial agreement to public commercial vendors. Comprehensive inland waterways information is available on Internet sites maintained by the National Weather Service and NOAA (<http://www.riverwatch.noaa.gov>) and the Corps of Engineers (<http://water.mvr.usace.army.mil>).

Hydrographic surveying and environmental data collection

Underwater site surveys are required to map bottom topography and cartographic features. Site surveying is conducted from either a surface-based platform or a submersible platform. Surface-based platforms are operated from ships, pontoons, or barges and generally operate on principles of reflected acoustic signal-processing using compression wave energy to determine water depths referenced to mean sea level. Surface position is determined using global positioning system (GPS) receivers that are operated either in a stand-alone mode or referenced to known latitude-longitude-elevation stations for increased accuracy (differential positioning). GPS navigation and surveying principles and

equipment were state of the art in the early 1990s, and are supplanting long-range radio navigation and radio beacon survey-positioning systems. GPS equipment development has focused on miniaturizing the receivers and providing a "total station" package to enable faster and more accurate survey results.

Having a highly accurate above-water reference datum provided by GPS technology allows for better positional accuracy beneath the surface. Echo-sounding (sonic) technology provides positioning data between the water surface and the bottom. Single-beam technology is being replaced by newer single-head multibeam (SHMB) technology and equipment. Several equipment manufacturers provide state-of-the-art equipment, and the Corps of Engineers is using those capabilities. For example, the Los Angeles District has acquired and used SHMB technology since the early 1990s for navigation works such as the Pier 400 harbor project (USACE 1999b).

The new International Hydrographic Office "Standards for Hydrographic Surveys" describe four orders of surveys, with Order 1 being the most stringent. Horizontal and vertical accuracy and maximum sounding line spacing for hydrographic surveys are established using formulas depending on average depth of investigation. For example, at a 99-ft (30-m)-depth coastal site, survey results should be accurate to at least 21 ft (6.5 m) horizontally with depth accuracy of 2 ft (0.63 m) (Harris 1999).

Submersible platforms consisting of remotely operated vehicles (ROV) or autonomous underwater vehicles (AUV) enable site surveying and topographic mapping in deeper water. The AUV systems do not have tethers, which eliminates tether hydrodynamic drag and results in faster survey operations. A downside of AUV technology is the lack of consistent and reliable energy sources for propulsion and onboard instrumentation. Development of acceptable sources (including batteries, fuel cells, and engines) is being undertaken by commercial and academic endeavors. An ideal AUV would have little or no input from an operator (i.e., would be preprogrammable) and would be capable of running its survey grid pattern at speeds of 1 to 2 m/sec while avoiding underwater obstacles. AUVs specifically designed for surveying operations have been developed which are capable of acoustically transmitting relatively accurate survey data but are not yet completely autonomous. Operator input is still required to adjust the survey instruments to match shifting seabed conditions and perform diagnostic functions for quality control (Bjerrum and Krogh 1998).

Environmental data collection is conducted to measure water quality data such as current velocity, density, temperature, and water chemistry profiles. Numerous equipment is available to obtain such data. To reduce the effect that the instrument itself has on the measured variable, new devices have been developed and are being used. For example, instead of using a standard current meter to obtain a velocity profile, electromagnetic current meters (EMCM) were developed based on the principle that a conductor (water) flowing through a magnetic field (current meter probe) produces a voltage proportional to its velocity. To further reduce the instrumentation effects, EMCM technology is being supplemented by newer noninvasive sensors based on acoustic emission principles. Acoustic

current meters, side-scan sonar, and seccan sonar devices allow current measurements to be taken at a single point or over a broad area. ROV and AUV platforms are also being packaged to provide multiple sensor instrumentation for bathymetric and environmental data collection (Sonsub 1999, Reson 1999, ORE 1999, Ultra Electronics 1999).

Geophysical investigations

Geophysical methods have evolved as major investigative tools for underwater site assessment and geotechnical exploration purposes. They are useful for detecting and delineating geological deformational features, including bottom and subbottom faults, scarps, rock and mud slides; for obstacle detection (Figure 25); landscape mapping (Figure 26); and for mapping other features critical to underwater foundation location and design.

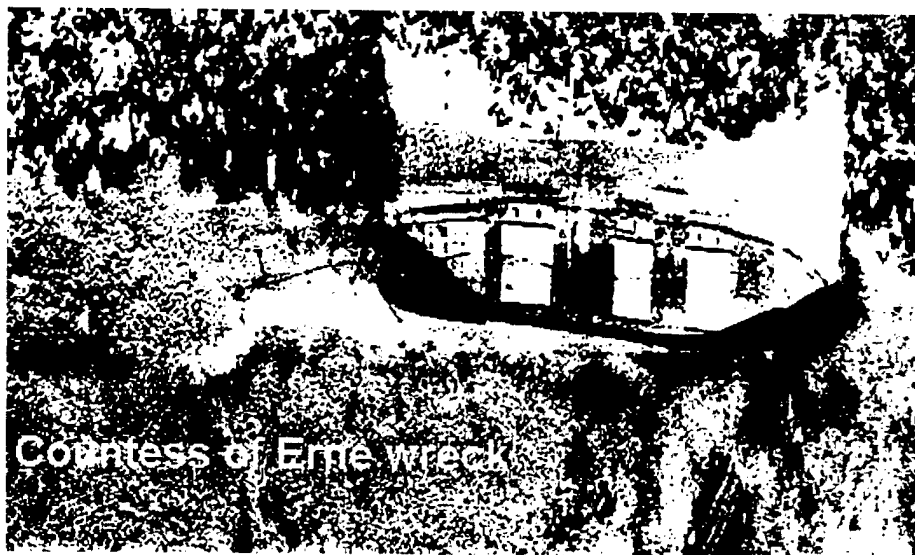


Figure 25. Widescan™ digital side-scan sonar image (after Ultra Electronics 1999) enables underwater obstacle detection

Offshore exploration technology relies on geophysical investigations to determine potential reservoir locations and to estimate potential yields. Near-surface geophysics accomplishes different functions at much shallower depths, but the principles are the same. Sound waves (acoustic and seismic), electrical resistance, electromagnetic signals, gravimetry, and radar principles and procedures are used in near-surface geophysical investigations, and each method is chosen for suitability depending on the intended function of the investigation (Beasley et al. 1997a, b).

Underwater acoustic and seismic methods are typically used for underwater site investigations; the required depth of investigation usually dictates which method is employed. Acoustic methods (subbottom profiling, side-scan sonar, and

echo-sounding) exploit compression wave propagation techniques and provide excellent results for determining stratigraphy just below the mudline. Seismic methods exploiting reflected or refracted shear wave propagation are useful for deeper investigations.

In general, the denser the subbottom material, the deeper the penetrability for these two methods. The implementation of geophysical acoustic methods and hydrographic surveying methods often overlaps due to the available equipment and technology. Multibeam echo-sounding and side-scan sonar equipment is capable of providing detailed landscape mapping of the seabed (or river bottom), which allows interpretation of geologic deformational features as well as providing physical positioning data for engineering surveys (Reson 1999, ORE 1999, Ultra Electronics 1999). Side-scan sonar methods are also used for quality control and monitoring of dredging and ocean disposal of dredged material. Mosher, Currie, and Sullivan (1997) investigated using side-scan sonar mosaics to monitor ocean disposal at depths to 822 ft (250 m).

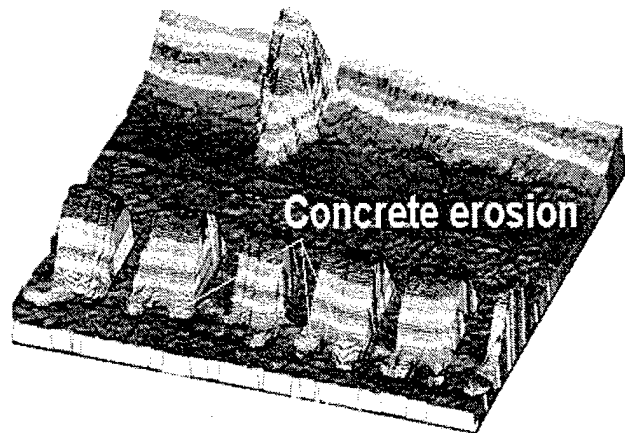


Figure 26. Bonneville Dam stilling basin mapping using SeaBat™ technology (after Reson, Inc. 1999)

Geophysical seismic investigations for offshore exploration use 3-D and 4-D (three- and four-dimensional time-lapsed) interpretation methods to map stratigraphy and reservoir deposits thousands of feet below the seabed. Advancing technology, such as ocean bottom cable systems, allows higher resolution and more accurate repeatability due to the fixed receiver (geophone and hydrophone) locations on and inside the seabed (Beasley et al. 1997a, b).

Other geophysical technologies commercialized in the offshore industry that may have application in foundation site characterization are the seismic-while-drilling (SWD) method, in which the advancing drill bit serves as the seismic source, and the wireline vertical seismic profiling (VSP) method (McMillin 1999). The major advantage of the SWD method is reduced costs during oil exploration drilling since the VSP method is not conducted in real time during the drilling operation.

High-resolution seismic data are also collected in shallow-water environments such as rivers and tidal areas. Typically, only 2-D interpretation methods are used, due to increased costs in conducting and interpreting 3-D high-resolution surveys. River bottom soil and rock stratification may be detected to depths greater than 100 ft (30 m) with 1-ft (0.3-m) vertical resolution when interpreting 2-D data. Advanced processing and interpretation techniques allow deeper depths with higher resolutions. Instead of ocean bottom cables, the receivers are located on

surface-towed platforms. Typically, a single channel system with a boomer source and a receiver is used, but multichannel systems are also used at more geologically complex sites.

Rivers and canals provide special challenges to geophysical investigations due to navigation restrictions and changing currents during the course of an investigation, as well as data interpretation of multiple mode reflection geometry caused by changes in river bottom and bank slopes (Toth, Vida, and Horwath 1997). As the available computer equipment and processing software becomes more adapted to shallow-water surveys, transition to high-resolution 3-D methods will likely become more commonplace in shallow-waterway project investigations.

Soil and Rock Engineering Properties

To obtain detailed subbottom information, geotechnical investigations are needed. Subsurface in situ testing and drilling and sampling techniques are required to determine or verify site geology, depth to bedrock, soil stratigraphy, etc. Preliminary assessment of the engineering properties of the soil or rock can be developed from the analyzed exploration data.

The soil/rock strength profile with depth is predicted from the interpreted soil/rock parameters (friction angle, apparent cohesion, etc.) resulting from the in situ and lab testing programs.

Recent trends suggest that in situ testing is strongly recommended to complement sampling (Reese and Isenhowe 2000). Samples are retrieved and tested to establish correlations to the in situ tests. The resulting soil/rock parameter selections and prediction of the strength and deformation profile with depth are input into the final engineering criteria for foundation design. Some discussions of the underwater geotechnical site investigation process are found in Heijnen and Vermeiden (1980) for an improved site foundation; Hitchings, Bradshaw, and Labiosa (1976) for a gravity-based foundation; and Angemeer (1972) and Audibert and Hamilton (1998) for a pinned foundation. Lunne and Powell (1993) provide details for offshore in situ testing applicability when evaluating engineering soil parameters for various soil and foundation types.

In situ methods

In situ testing allows evaluation of engineering properties without physically retrieving samples, often without requiring a predrilled borehole. Geophysical techniques and equipment may be used to acquire geotechnical engineering properties. Geotechnical devices frequently used for underwater in situ testing are the remote vane shear device, the cone penetrometer, and the pressuremeter. These methods were initially developed for land-based applications but have been adapted for underwater site characterizations. The remote vane was the first in situ

tool to be routinely used in the early 1970s, followed by the cone penetrometer and the pressuremeter (Briaud, Riner, and Ohya 1984).

Geophysical methods. Geophysical investigations can provide overall information regarding engineering properties at the foundation site such as anticipated sediment type, stratigraphy, depth to bedrock, and dynamic properties. Preliminary soil index properties may also be inferred, and studies have shown correlation between geophysical measurements and engineering properties of the subbottom (Mahmood and Hough 1978; Kutter, Arulanandan, and Dafalias 1979; Fountain 1979; Haynes et al. 1993; Nauroy and Meunier 1993; Stokoe and Rosenblad 1999).

Acoustic methods (sonar and subbottom profilers) provide no direct engineering properties information. Shear wave methods furnish the shear modulus, G , of the subsurface material whether it be soft mud or solid rock. The shear wave geophysical method is matched to the site and its required investigation depths. For example, intrusive methods include seismic cone penetration (SCPT) and borehole (crosshole, downhole, and suspension) logging techniques. Nonintrusive methods include refraction surveys and surface wave profiling (Stokoe and Rosenblad 1999). Other marine geophysical methods to obtain engineering properties include gamma logging for bulk density measurements (Perlow and Richards 1972) and electrical resistivity to measure index properties of soils (Kutter, Arulanandan, and Dafalias 1979).

Vane shear device. Shear strength less than about 2 tsf (200 kPa) in cohesive sediments is measured using a vane shear testing system in which the unit is either deployed over the side of a surface vessel or lowered through a drill string (wireline technique) (Lunne and Powell 1993). As the unit settles onto the bottom on its attached template or reaches the borehole bottom, a vane penetrates undisturbed soil. The vane is then rotated, and the soil's shearing resistance (measured by a rotational strain gauge transducer) is correlated to its undrained shear strength.

Deep-sea vane shear strength measurements were made in the early 1960s from manned research submersibles such as the U.S. Navy's *Trieste* and *Deepstar 4000*, Woods Hole's *Alvin*, and Lockheed's *Deep Quest* (Perlow and Richards 1972; Inderbitzen and Simpson 1972; Hirst, Richards, and Inderbitzen 1972). Their operational depths ranged to about 20,000 ft (6,080 m), but sampling depths were limited to about 5 ft (1.5 m) below the sea bottom (Noorany 1972). Tethered testing platforms placed directly on the ocean floor were developed by the late 1960s and early 1970s (Richards et al. 1972). The *RUM* was an unmanned tethered tracked bottom crawler that was instrumented with a vane shear device and other sampling tools and had an operational depth of 6,000 ft (1,824 m) (Anderson et al. 1972). The Navy's *DOTIPOS* tethered platform vane could penetrate 10 ft (3 m) below the mudline (Noorany 1972).

The offshore industry began using vanes located on tethered submersibles that were operated remotely from surface drilling ships using the wireline technique (Perlow and Richards 1972; Doyle, McClelland, and Ferguson 1971). Deploying

the vane device from a platform resting on the ocean bottom later became more common (Dutt et al. 1997).

New-generation remote vane shear devices such as the *Halibut II* (Figure 27) have been developed to reduce testing setup time, to increase the incremental penetration depth, and to perform cyclic testing (Dutt et al. 1997). Cyclic in situ vane tests are conducted to observe the effects of fully reversed, two-way loading. The vanes are torqued for 5 to 11 cycles, and the plotted results are useful for design calculations required for structures undergoing dynamic loading such as wave impact. Remote memory units attached to the vane allow for better-quality data collection.

Cone penetrometer. Cone penetrometer technology, which has seen many successful land-based applications during the past decades, has also been adapted to underwater site characterization. This technology is applicable to both cohesive and cohesionless soils. The penetrometer delineates stratigraphy, provides penetration resistance data that can be empirically correlated to soil strength and deformation parameters, measures pore-water pressure, and is adaptable for

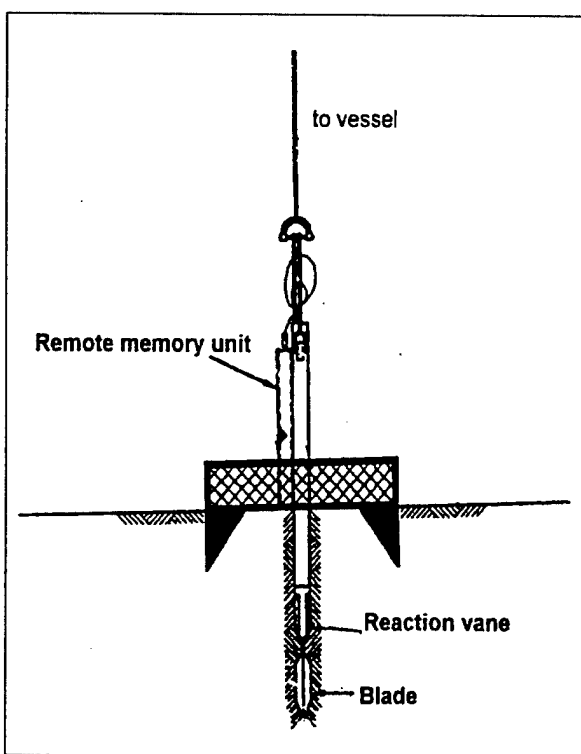


Figure 27. Remote vane shear device (after Dutt et al. 1997)

applications requiring specialized sensors, such as resistivity and seismic geophysical investigations. Soil sample retrieval may also be achieved using modified cone penetrometer equipment. Particularly useful is the pore-water pressure measurement capability (piezocone or PCPT). Determining subsurface pore-water pressures and indirectly assessing in situ permeability is important in understanding the presence or absence of pore-water gradients in the subsurface soil layers, which may cause problems with the future foundation's static and dynamic stability. Lunne and Powell (1993) state that the piezocone is the most important in situ tool for offshore use.

The cone penetrometer test (CPT) has been used worldwide during the 20th century as an onshore site investigation method that is relatively fast and economical and produces consistent data (FHWA 1978, ASTM 1991). Various changes and technical improvements have been made, but the CPT remains particularly useful in

assessing soil classification and strength while probing on-the-fly. Offshore CPT provides more accurate soils data than just sampling alone (Marr and Endley 1982).

Underwater CPT for deep-ocean exploration was conducted in the early 1970s on tethered submersibles such as the *Deep Quest* (Hirst, Richards, and Inderbitzen 1972), the *Alvin* (Perlow and Richards 1972), and the *DOTIPOS* (Noorany 1972). Penetration depths were limited to about 10 ft (3 m). A specially designed manned submersible (*Mission*) for geotechnical site investigation was used for the Eastern Scheldt project in the late 1970s. The *Mission* held two men breathing supplied surface air in a submerged steel capsule that was lowered to the seafloor. A hydraulic ram inside the capsule pushed the CPT and soil sampling rods 100 ft (30 m) below the mudline (Heijnen and Vermeiden 1980).

From the 1970s, the CPT has been conducted either from seafloor-based systems (similar to the one shown in Figure 28) or down-the-hole (wireline) systems. Some of the seafloor systems included McClelland's *Seacalf* and *Stingray* (Ferguson, McClelland, and Bell 1977); Fugro-McClelland's *Seasprite* (Lunne and Powell 1993); the U.S. Navy's *XSP-40* (Beard and Lee 1982); European systems by A.P. van den Berg (*ROSON*) and the Danish Geotechnical Institute's *SCORE* (Lunne and Powell 1993); and *TSP* (Humphrey and Adams 1995). The *TSP* system was designed to operate in water depths to 9,900 ft (3,000 m) while probing to soil depths of 230 ft (70 m).

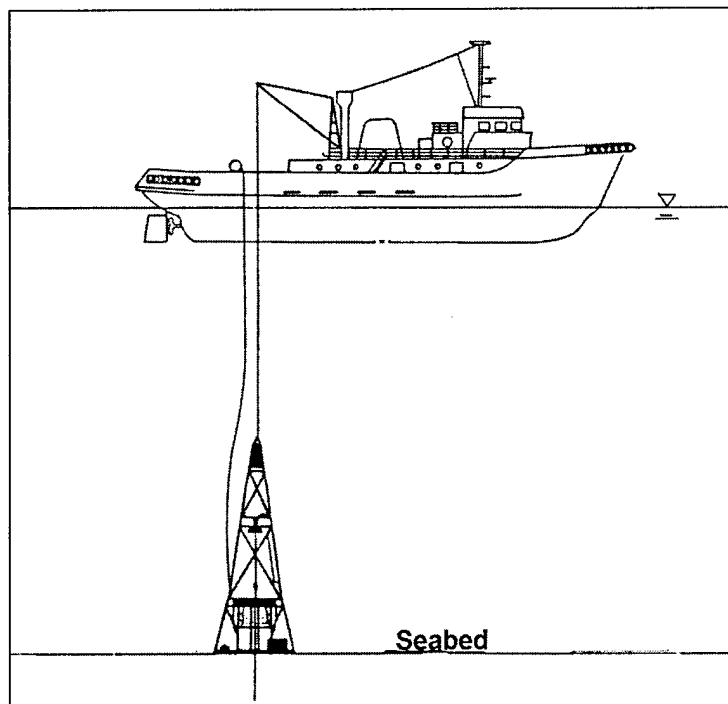


Figure 28. Remote CPT operation from a surface vessel (after James 1987)

The wireline technique allows multiple downhole tools and samplers to be used in the same borehole. One example of a wireline system using multiple downhole tools is McClelland's *Swordfish*, in which the CPT probe is advanced

10 ft (3 m) below the borehole base (Meyer, Harman, and King 1982). McClelland's *Dolphin* system stores all data in its memory module and does not have an umbilical cable (Focht, Johnson, and Rivette 1986; Lunne and Powell 1993). The *Dolphin* system also includes a piezometer probe to measure pore-pressure dissipation. The remote memory module stores up to 135 min of pore-water pressure dissipation data. After the tool is retrieved, the data are downloaded and analyzed (Preslan and Babb 1979, Dutt et al. 1997). Fugro-McClelland's *WISON* and Geoclean's *MASCOT* systems operate in the shallower North Sea environment (Lunne and Powell 1993).

The U.S. Navy recently developed a dynamic penetrometer system that is dropped over the side of a surface vessel, falls through the water column, and surficially penetrates the seafloor while simultaneously measuring probe deceleration using Doppler sound imaging. Correlations between deceleration time and relative density of cohesionless soils have been made (Orenberg et al. 1996).

The seismic cone penetration test has been performed from seafloor-based and surface platforms since the late 1980s (Lunne and Powell 1993). Penetration depths on the order of 200 ft (60 m) have been achieved (Stokoe and Rosenblad 1999).

Pressuremeter. Another in situ testing tool is the pressuremeter, which was introduced (onshore) by Kogler and Scheidig in the 1920s (Parkhill 1998) and was further developed by Menard in the 1950s. The pressuremeter operates on the principle of expanding a vertical cylinder horizontally into the surrounding soil while measuring the volumetric strain and applied pressure. Correlations are made to determine the soil's in situ horizontal stress and shear strength. The pressuremeter test (PMT) is particularly suited for determining resistance to induced static and cyclic lateral loads on piles and drilled shafts (Bowles 1988; Briaud, Smith, and Meyer 1983; Briaud, Riner, and Ohya 1984). The major limitation of obtaining quality data from the PMT is the requirement for a carefully prepared borehole (Bowles 1988). To overcome borehole preparation problems, the self-boring pressuremeter test was developed.

Offshore pressuremeters have been developed by various companies (Reid et al. 1982; Suyama, Ohya, and Imai 1982; Lunne and Powell 1993) and are generally wireline self-boring.

Drilling and sampling

Geotechnical drilling and sampling operations are typically required to supplement the in situ test data and to extract samples. Often a boring is advanced for purposes of obtaining information, such as soil and rock classification from visual inspection or blow counts from the standard penetration test. However, the primary advantage of the drilling and sampling operations is to retrieve undisturbed or disturbed samples from the subbottom for subsequent laboratory testing. Before, during, or after the samples are retrieved, various types of devices

may be advanced in a cased or an uncased borehole for purposes of obtaining in situ geotechnical information about the subbottom. For example, smaller diameter wireline piezocone penetrometers have been developed specifically for use inside boreholes. The PCPT is mechanically extended past the borehole opening to probe virgin soil for the purposes of obtaining pore-water pressure, which has been correlated empirically to in situ permeability as well as the standard data obtained by the CPT ahead of the advancing drilled borehole (Dutt et al. 1997).

Underwater soil sampling techniques and equipment development have generally followed those of land-based methods. Prior to the 1940s, the offshore industry did not conduct preliminary site investigations. The first marine foundation site investigation for an offshore structure was drilled from a fixed platform in 30-ft (9-m) water depth in the Gulf of Mexico in 1947. As the oil platform water depths increased, so did the capacities of the exploratory drill platforms, and they evolved from fixed platforms to mobile drill rigs on barges. Perhaps the most challenging location was the Bay of Fundy, where fixed-piston samples of soft marine clays were recovered from depths of 326 ft (99 m) while coping with 25-ft (8-m) tides and currents up to 8 fps (2.4 m/sec). This type of exploration method using conventional land-based sampling techniques was used for most subsea locations at water depths to about 200 ft (60 m) until the year 1962 (Focht and Kraft 1977).

In 1962, an oil company conducted a program to gather geotechnical data along the outer Continental Shelf to depths of 600 ft (180 m). Instead of a floating barge, a purpose-built boat was outfitted with a large drilling rig. Special lightweight 3-in.-diam drill pipe and wireline sampling tools were developed to penetrate 300 ft (90 m) below the seabed. The wireline samplers enabled sample retrieval without bringing the drill pipe sections back to the drill rig (Focht and Kraft 1977).

In the 1970s and 1980s the offshore industry moved into deeper waters in the Gulf of Mexico, and geotechnical site investigation techniques and equipment were developed to meet those challenges. Investigations at seafloor depths of 2,900 ft (880 m) were conducted. The Shell "Mensa" project required investigations at a seafloor depth of 5,315 ft (1,616 m). Up to the mid-1990s, total penetration (water and soil column) of geotechnical drilling and sampling for the majority of the Gulf of Mexico projects had been limited to about 6,600 ft (2,010 m) based on available surface vessels. In 1997, a converted dynamically positioned diving support semisubmersible was mobilized to provide site investigation capabilities for deeper sites. It allowed drilling to water depths of 9,900 ft (3,000 m) and had a total drillstring capability of 11,840 ft (3,600 m) (Dutt et al. 1997).

A recent Gulf of Mexico program sponsored by several oil companies and the National Science Foundation collected geotechnical data in water depths to 7,700 ft (2,340 m). A series of in situ vane shear tests, pore-pressure measurements, deepwater coring, sample retrieval, and laboratory tests were conducted (Sea Technology 1999).

The methods of underwater sampling and selection of equipment are arbitrarily divided into two general categories depending on the water depth and sampling depth: shallow sampling depths to 100 ft (30 m) below the mudline, and deeper sampling depths. Most geotechnical sampling has been done from the water surface using a fixed or mobile platform (Noorany 1972). Numerous systems employed for soil sampling from submersible seabed platforms in deeper water were described by Tirey (1972) and Hironaka and Green (1971). Most of the platforms had submersible hydraulic rotary drills, but some had submersible hydraulic vibratory hammer-type drills predating submersible vibratory pile hammers by almost 2 decades.

Sampling tools specifically used for underwater sampling include diver-held samplers, the Boomerang corer, box corer, gravity corers, vibratory corer, wireline samplers, and dredging grab samplers (U.S. Army Corps of Engineers 1996). In general, the diver-held samplers, corers, and grab samplers are used for shallow sampling penetrations. For deeper penetrations, wireline samplers are used

(Noorany 1972). An illustration of one type of wireline sampling configuration is shown in Figure 29. Tirey (1972) discussed a range of types of submersible, remote, and diver-operated sampling equipment, including rotary and vibratory drilling tools. A noteworthy comment was that regardless of the sampling tool, no feasible sampling method completely eliminates sample disturbance. Furthermore, the disturbance of samples obtained at nearshore and offshore sites was generally greater than the disturbance of samples obtained on land because of the greater difficulty of obtaining quality samples. Reese and Isenhower (2000) reported that wireline sampling techniques caused more disturbance than the worst onshore sampling methods when applied to the offshore environment.

The popular onshore ASTM D 1586 standard penetration test (SPT) consists of dropping a 140-lb (308-kg) weight 30 in. (0.76 m) onto a drill string with a 2-in. (51-mm)-diam sampling tube attached at the bottom.

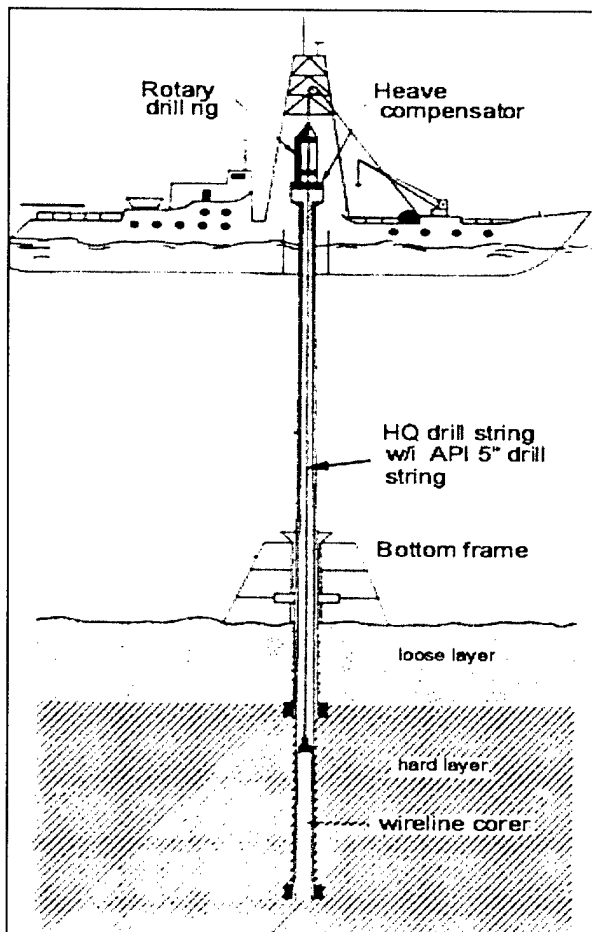


Figure 29. Offshore wireline sampling technique (after CLAROM 1994)

The number of blows required to penetrate the sampler 12 in. (300 mm) into the borehole bottom is correlated to the soil's classification and strength. The recovered sample may be used for laboratory testing purposes (Acker 1974).

The SPT may be used for nearshore and river locations where the wave heights are minimal, but the offshore industry does not traditionally use the test because the fall height of the hammer cannot be accurately controlled unless the ship heave is compensated (Hitchings, Bradshaw, and Labiosa 1976). Bazett and Foxall (1972) compared and discussed SPT blow counts from a natural riverbed and subsequent underwater-placed fill from a spud-barge platform. Babcock and Miller (1972) compared vibracore sampling to SPT data and found good correlations in low blow count soil deposits.

Offshore rotary core sampling is generally used only for identification purposes, but wireline tube sampling is generally used for the full range of soil types and strengths (Hight 1993). The two most common types of tube sampling methods are push sampling and piston sampling. Most push sampling uses 3-in. (76-mm)-diam thin-walled Shelby tubes for undrained soil strengths up to about 6 tsf (600 kPa). Thicker walled tubes are used to recover dense sands and hard clays. Tube liners are used for upper sediments with little or no shear resistance. Piston samplers are used in soft and firm clays when sample retention would be a problem (Hight 1993), but piston sampling is not commonly used offshore in the Gulf of Mexico (Jeanjean, Andersen, and Kalsnes 1998). McCoy (1972) discussed piston sampling characteristics in the North Atlantic.

Another offshore sampling method uses large-diameter borehole technology to obtain bulk samples with techniques and equipment similar to those used onshore. Larger boreholes (diameter between 1 and 10 ft, 0.3 and 3 m) enable larger sample sizes for better quality classification and testing. Unlike dredging samples that are obtained from shallow depths, large-diameter boreholes allow sampling at depths approaching 400 ft (120 m) (Schwank 1997).

Laboratory testing

To the extent that site assessment is not complete until all engineering design parameters are obtained, many of the retrieved soil/rock samples undergo laboratory testing. The laboratory testing program should be tailored to obtain engineering parameters for preliminary design of the intended foundation system. For example, both monotonic and cyclic shear strength profiles are needed for determining the holding capacity of suction caissons (Jeanjean, Andersen, and Kalsnes 1998). Sangrey, Clukey, and Molnia (1972) discussed methods to obtain strength profiles from disturbed or remolded samples.

The types of soil (and/or rock) laboratory tests conducted for underwater foundations should accomplish the following objectives (Sullivan, Wright, and Senner 1980): (a) material identification and classification, (b) behavior under anticipated field levels of stress and strain, (c) compressibility characteristics

under sustained loading, and (d) stress-strain characteristics and pore-pressure response under cyclic loading.

The tests on soil materials should include index properties (submerged unit weight, Atterberg limits, mineralogy, grain-size analysis, moisture content, chemical composition, and specific gravity). Strength tests should be conducted using either "undisturbed" or remolded sample specimens. Unconsolidated-undrained (UU) tests can be conducted to assess short-term behavior, whereas consolidated-drained (CD) or consolidated-undrained (CU-bar) tests with pore-pressure measurements are needed to assess the long-term behavior.

Conventional soil strength tests imply static (monotonic) loading conditions and include direct simple shear tests, unconfined compression triaxial tests, UU triaxial tests, miniature shear vane, Torvane, fall cone, and hand penetrometer, among others. Computed strength and stiffness parameters (c_u , ϕ_u , E_u) from these test data establish the undrained shear strengths of the soil specimens.

To determine normalized and effective stress strength parameters, CD (direct shear) tests or isotropically consolidated (CIU-bar) and anisotropically consolidated (Ck_oU -bar) consolidated-undrained monotonic triaxial compression tests are conducted (Sullivan, Wright, and Senner 1980). For the triaxial tests, pore-pressure measurements are required to determine the effective stress states (Audibert and Hamilton 1998). The CD test, by definition, yields effective stress strength and stiffness parameters (c_u' , ϕ_u' , E_u').

Less conventional tests are often required as an attempt to replicate anticipated dynamic and cyclic loading conditions, such as the loading conditions caused by wind, wave, and seismic loads. Cyclic anisotropically consolidated-undrained (C_yCK_oU) triaxial compression and cyclic direct simple shear tests are typically performed to define the cyclic stress-strain behavior of soils. A laboratory test to determine the shear wave velocity and maximum shear modulus is conducted using a resonant column device (Jeanjean, Andersen, and Kalsnes 1998).

To determine stress history of cohesive soils, consolidation tests are routinely conducted. The overconsolidation ratio can be determined using the constant strain and incremental loading oedometer tests. Consolidation test data are used to determine the time-settlement curve and void ratio-effective stress relationships (Sullivan, Wright, and Senner 1980).

Uncertainty in laboratory test data interpretation arises from the effects of soil heterogeneity, sample disturbance, in situ stress history and loading conditions, and testing accuracy. For example, in a layered soil subjected to loading condition imposed by a structure (see Figure 30), a triaxial compression test would best simulate the failure conditions immediately under the structural footing, whereas the direct shear test would better simulate the loading conditions at the deeper portion of the failure plane as it is passing through the layered soil. Likewise, as the slip surface daylight at some distance from the footing, the triaxial extension

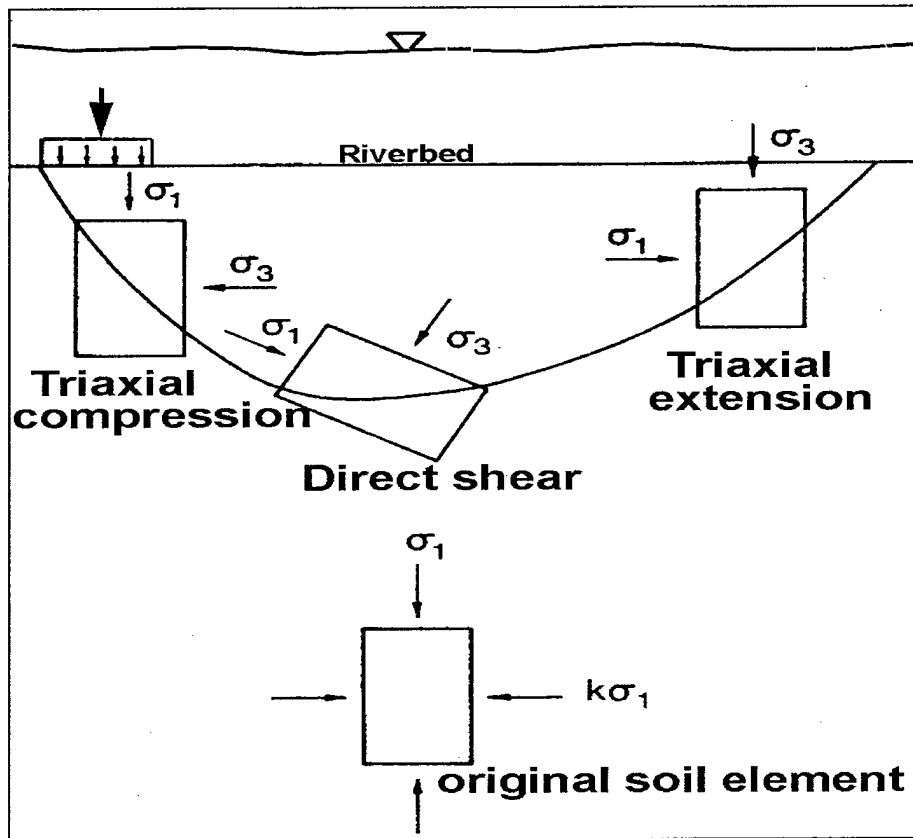


Figure 30. Sketch of idealized model of in situ stresses (after Sullivan, Wright, and Senner 1980)

test would better replicate the loading conditions due to the reversal of principal stresses.

One method for assessing the reliability of laboratory data uses soil parameter empirical correlations. Sullivan, Wright, and Senner (1980) discussed these uncertainties and suggested correlations for comparison. For example, an estimate of the remolded shear strength for an overconsolidated clay can be made from its liquidity index. The ratio of normally consolidated clay shear strength to effective overburden stress is related to its plasticity index. Fine sand's friction angle is related to its relative density and voids ratio. However, it should be emphasized that these empirical correlations should not be substituted for a detailed laboratory investigation.

In addition to the strength and deformation characteristic for rock, the rock material properties should include porosity, density, sonic velocity, permeability, ductility, strength, slake durability, hardness, and thermal properties. Rock mass strengths are rated by either the RMR or Q systems (Goodman 1989).

Offshore foundation material (soil and rock) databases are available that can be used for planning the laboratory investigation and interpreting the test results.

These databases are especially useful for planning cyclic loading test programs, such as suggesting methods to reduce the number of site-specific cyclic tests. Andersen and Jostad (1999) list references for available databases of North Sea and Gulf of Mexico clay soils. Numerous other references to site-specific soil/rock characteristics related to geotechnical foundations are available in the literature and include the following:

- Sabine River delta (Grosch and Reese 1980)
- Offshore Florida calcareous sands (Dutt et al. 1986)
- Offshore Southern California (Mahmood and Reifel 1978)
- Mississippi Delta (Gulf of Mexico) weak sediments (Shephard, Bryant, and Dunlap 1978; Booth and Garrison 1978)
- Gulf of Mexico underconsolidated clays to dense sands (Stockard 1979)
- Alaska coastal waters (Sangrey, Clukey, and Molnia 1979)
- Hawaii calcareous sands (Lu 1986)
- Offshore Cuba coral sands (Puech, Bustamante, and Auperin 1990)
- Offshore Brazil calcareous soils (Ping et al. 1984)
- Offshore Australia calcareous sands and sandy silts (Parkin et al. 1990; Tan, Parkin, and Yee 1990; Randolph et al. 1996)
- Borneo marine clays (Gemeinhardt and Yan 1978)
- Offshore India (Stockard 1986)
- Offshore China dense sands (Tsien 1986)
- Arabian Gulf hard clays, dense sands, rocks, and coral (Tagaya et al. 1979; Settgast 1980; Stevens, Wiltsie, and Turton 1982; Vines and Hong 1984)
- Gulf of Suez calcareous sands and silts (Dutt and Cheng 1984, Dutt and Teferra 1986)
- North Sea stiff clays (Heerema 1979)
- North Sea soft clays (Karlsrud and Nadim 1990)
- North Sea dense sands (Jardine and Overy 1996, Zuidberg and Vergobbi 1996)
- North Sea boulder clays (Fox, Parker, and Sutton 1970)
- North Sea chalk (Vijayvergiya, Cheng, and Kolk 1977; Davie, Ehlers, and Antes 1978)

4 Design Considerations

Foundation Types

Numerous foundation types are constructed onshore (as land-based construction), and they broadly fall into either the “shallow” or “deep” foundation category. By definition, the shallow foundation embedment depth is less than its minimum lateral dimension and includes spread footings and mats. The deep foundation embedment depth is much greater than its minimum lateral dimension. Examples include piles and shafts (Herrmann, Raecke, and Albertsen 1972). Since the terms shallow and deep may be confusing when applied to underwater foundations, those terms will not be used for the remainder of this document.

Underwater foundations may be classified into one or more of the following general categories: *improved-site*, *gravity-based*, and *pinned*. Improved-site foundations include those that are dredged, excavated, backfilled, leveled, and/or site-modified. Gravity (or gravity-based) foundations include those that distribute the structure’s loads over a soil/rock area wide enough to adequately resist the imposed loads. Pinned foundations distribute the structure’s loads deeper into the soil/rock materials. Table 1 provides a generalized description of foundation types for selected case histories and includes prominent foundation preparation/construction features.

Certain structures are generally associated with a given foundation type. For example, most immersed tube tunnels depend on site-improved foundations. Likewise, the vast majority of offshore oil production platforms in the Gulf of Mexico depend on pinned (driven-pile) foundations. However, foundation types may also be combined for certain structures. For example, some gravity-based foundations such as concrete bridge piers require site improvements prior to float-in or lift-in placement of the piers. Likewise, some gravity-based foundations include piles. Some suction-pile foundations imitate both gravity-based and pinned foundations in that they may be designed to distribute their ballasted weight over a large lateral area during vertical penetration.

For gravity and pinned foundations, an improved-site foundation may or may not be needed. Based on published case histories, typical deep offshore practice does not include improving the site prior to installing either gravity or pinned foundations. However, for shallow-water gravity foundations, some type of site improvement is usually provided. Two examples of Corps projects that will use

Table 1

Underwater Foundations

Foundation type	Example	Site	Foundation Preparation/Installation
<u>Improved-site</u>	BART Tunnel ¹ Oosterschelde ¹	Soft clay Sands/silt	Dredged trench, select gravel backfilled and screeded level Dredged undesirable materials, vibrocompacted sand, engineered geotextile mats placed under float-in concrete piers
	H. Keenleyside Dam ³	Sand/gravel	Removed silt sediment, free-fall fill placement of gravels and glacial till
<u>Gravity-based</u>	Akashi Bridge ⁴ Confederation Bridge ⁵ Brooklyn Bridge ⁶	Soft sediment Siltstone Soft Sediment	Float-in steel caissons sunk on dredged site with compacted gravel backfill 6,000-ton float-in concrete pier base sunk on dredged site Timber caissons sunk to bedrock by manual excavation in compressed air chambers
<u>Pinned</u>			
Soil displaced (Driven or suction piles):			
	Conoco's TLP Ursa ⁷ Statoli's Europe ⁸ Tappan Zee Bridge ⁹	Sand/clay Dense sand Layered soils	417-ft-long, 96-in.-diam piles driven with underwater hammer 36-ft-diam buckets (steel skirts) penetrated by suction H-piles driven through float-in concrete caissons setting on 5-ft gravel bed
Soil excavated (Drilled or augered shafts, piles, and caissons):			
	Boston Tunnel ¹⁰ Braddock L&D ¹¹ Pocomoke Bridge ¹²	Clay Claystone Dense sand	6-ft-diam drilled shafts supporting immersed tube sections 78-in.-diam drilled shafts to bedrock support float-in gated dam 7-in.-diam cased pinpiles drilled to replace rotted timber piles

¹Bay Area Rapid Transit, San Francisco (Kussel 1986)²Eastern Scheldt Storm Surge Barrier, The Netherlands (DOSBOUW 1987)³British Columbia (Bazett and Foxall 1972)⁴Akashi Straits, Japan (Kajima Corporation 1998)⁵Northumberland Strait, New Brunswick, Canada (*Serwick News* 1992)⁶East River, New York (Delaney 1983)⁷Tension Leg Platform, Gulf of Mexico (Digre, Kipp, and Hunt 1999)⁸Jacket Platform, North Sea (Baerheim, Hoberg, and Tjelja 1995)⁹Hudson River, New York (New York State 1999)¹⁰Central Artery Project, Boston (Brudno and Lancellotti 1992)¹¹Corps of Engineers, Pittsburgh District (ASCE 1999a)¹²Bascule pier drawbridge, Pocomoke River, Maryland (Bruce 1992)

site improvement in conjunction with pinned foundations are the Corps' Braddock and Olmsted Dams. At Braddock Dam, the float-in precast elements will be ballasted and attached to pinned foundation (drilled shafts). Softer sediments will be dredged prior to and during construction (as needed). At Olmsted Dam, the lift-in precast elements will be placed on pinned foundation (driven-pipe piles). Site improvement will include dredging, sand vibrocompaction, and articulated mat placement (Personal Communication, 2 March 2001, Dale E. Berner, consulting engineer, Ben C. Gerwick, Inc., San Francisco, CA).

Improved-site foundations

An improved-site foundation modifies the existing soil (or rock) in the seabed or riverbed. Methods include preloading, dredging, backfill, leveling, consolidation, or other engineered modifications. Figure 31 illustrates one example of an improved-site foundation. In the case of immersed tunnels, pipelines, breakwalls, dry docks, and some float-in/lift-in structures, an improved site may be the only foundation system needed.

Dowse (1979) described an innovative improved-site methodology employing self-supporting (hydrostatic pressure) sand islands for foundations for oil platforms in water up to 200 ft (61 m) deep. Basically, an impervious membrane is filled with dredged sand, and the water is then pumped out. Such hydrostatically supported confined-pressure sand islands are capable of supporting surface loads that are proportional to the drained shear strength of the sand. Each island can be built within a 2-week period depending on the specific design. Disadvantages are the amount of dredging required to supply the sand fill, the limitations of a soft soil underbase, and the construction handling and placement of the impervious membrane.

Numerous other examples of improved-site foundations are noted in this report. Dredging, fill placement, consolidation, soil improvement, and mat reinforcing are some of the methods used during construction of an improved-site foundation.

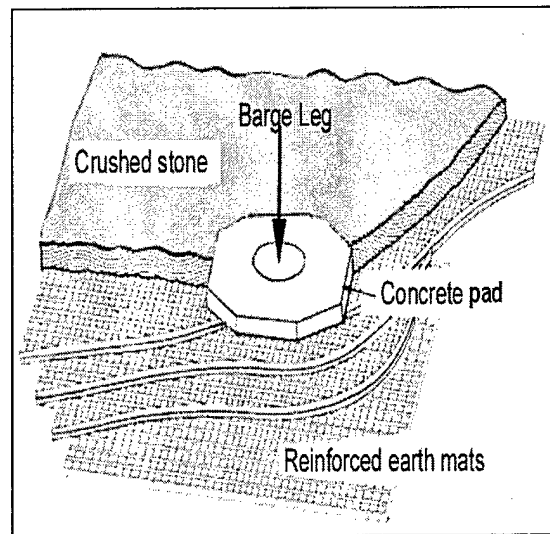


Figure 31. Example of an improved-site foundation (after Abbott and George 1989)

Gravity-based foundations

Gravity-based foundations are broadly classified because the load-carrying capacity is primarily determined by the size and weight. These foundations are similar in function and design to the land-based foundations that are classified as “shallow,” since their embedment depths are generally less than their lateral dimensions. Typical gravity foundations are constructed of precast concrete elements that are either floated or lifted in and allowed to sink into the mudline or onto an improved-site foundation. Once installed, the underbases may be filled with cast-in-place concrete or grout via underwater tremie tubes or other type of ballasting materials. For example, the main pier bases supporting the Confederation Bridge in Canada are lift-in precast concrete elements resting on a tremied concrete bed over mudstone and siltstone layers. Each of the forty-four 6,000-ton pier bases rests on a ring footing of tremied concrete having an approximate diameter of 67 ft (20 m) and an approximate thickness of 6 ft (1.8 m).

Smaller gravity-based foundations have been constructed of steel instead of concrete. For example, offshore foundations for wind turbines have been constructed of steel gravity foundations that are basically steel boxes sunk to the seabed and filled with olivine (DWTMA 1998). The steel box size was designed to be 42 by 42 ft (14 by 14 m) with a weight of about 90 tons (100 metric tonnes) for a water depth to 30 ft (9 m).

Wooden timbers form the gravity base of the Brooklyn Bridge, which was completed in 1883. Two wooden box structures, one on each side of the river, were weighted with stones to enable contact with the underlying bedrock (Delaney 1983). Each box weighed about 16 million pounds (7.3 Mg) with a footprint of about 17,000 sq ft (1,579 sq m). Examples of present-day wooden gravity-based foundations are rare, since the preferred materials are either concrete or steel.

For offshore structures, submerged gravity-based foundation weights may approach 1 million tons. A modification of the gravity base concept is the addition of suction skirts around the foundation perimeter to help resist lateral, tensile, and cyclic loads imposed on the gravity foundation.

Pinned foundations

Pinned foundation embedment depths are greater than the minimum lateral dimension, and include piles and drilled shafts. They are installed to achieve greater lateral stability, to provide greater uplift resistance, to provide greater resistance to scouring compared with gravity-based foundations, and to minimize the potential for differential settlement of certain critical structures. The two general types of pinned foundations are those that displaced soil (that is, piles) and those in which soil was removed (drilled shafts). Pinned foundations are constructed of wood, steel, concrete, grout, stones, or polymer materials that are pinned into the seabed or river bottom by application of driven or drilled-energy sources. Techniques and equipment for deep-water (typically more than 300-ft)

shaft, pile, and caisson construction are generally found within the offshore oil-drilling industry.

Displaced soil. These methods include driving piles into the soil or weak rock by impact, vibratory oscillation, hydraulic pushing or jacking methods, rotating, jetting, or suction techniques and equipment. Hydraulic pile-driving is virtually noiseless and vibration-free, while pile jetting is a time-saving pile-driving method (Tsinker 1988). A brief discussion of several types of displaced-soil piles follows.

- **Driven preformed piles.** Examples of driven preformed piles include tubular steel (open or closed end pipe) piles, solid steel (H-, X-, A-, or sheet) piles, timber piles, precast (prestressed and/or reinforced) concrete piles, and polymer piles. Figure 32 illustrates an underwater impact-driven pile installation procedure. The advantages of driven preformed piles include the following:

- a. The pile materials are preformed and subject to fabrication and site (quality control/quality assurance) inspection.
- b. The pile materials can withstand high bending and tensile stresses.
- c. Numerous choices for pile materials, equipment, and installation procedures are available.
- d. Onsite fabrication permits very long lengths of piles for site-specific conditions.

The disadvantages of driven preformed piles include:

- a. Above-water noise and vibration may be excessive.
- b. Soil disturbance may affect surrounding structures.

- **Cast in situ piles.** Examples of cast in situ piles include stone columns (Cemcol), compacted concrete pile (Vibro-Franki, not commonly placed underwater), Atlas screw pile (not yet commonly used underwater according to Reese and Isenhower 2000), and micropiles (which are grouted minipiles or pinpiles installed without drilling).

Figure 33 illustrates an onshore Atlas screw pile installation procedure. The advantages of cast in situ piles are:

- a. No need for permanent casing.

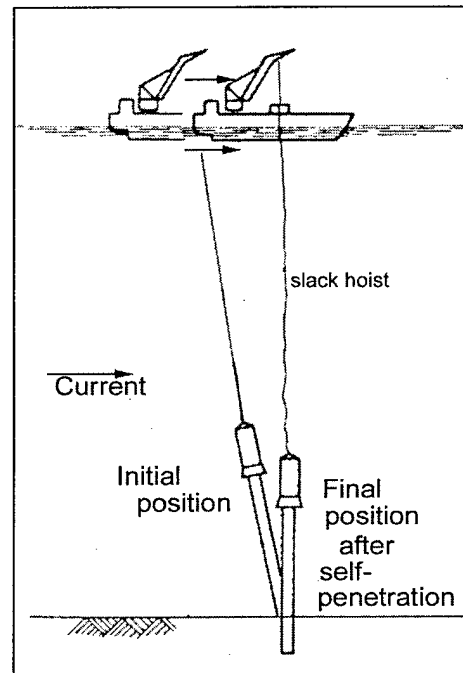


Figure 32. Underwater impact-driven pipe pile installation without a template (after Jansz and Brockhoff 1979)

- b. May be more economical for specific sites.
- c. No cuttings and, hence, no disposal of cuttings required.

The disadvantages of cast in situ piles include:

- a. Limited experience in underwater environments and (for certain proprietary systems, such as Cemcol, Atlas, Vibro, Franki, Vibrex, and Fundex) extremely limited specialized contractor experience.
- b. Cannot be extended above riverbed without casing and special procedures.

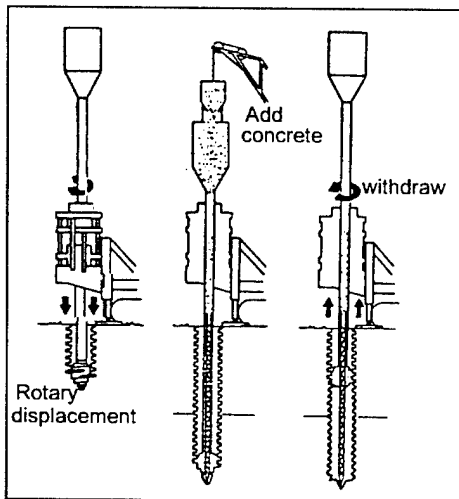


Figure 33. Cast in situ onshore pinned foundation (Atlas screw pile method) (after Hollingsworth and Imbo-Burg 1992)

A special case of the cast in situ pile is represented by the soil and grout cast in situ foundation, which is generally designed as an improved-site foundation instead of pinned. This type of foundation improvement is used primarily for onshore applications, although deep cement mixing has been used offshore (although primarily outside the United States).

Two examples of the soil and grout cast in situ foundation improvement include jet-grouted columns (grout injected into soil from high-pressure jets) and mixed soil columns (grout mixed with soil, also called deep cement mixing). The advantages and disadvantages mirror those for cast in situ, as previously noted.

- **Driven cast in situ piles.** An example of the driven cast in situ pile includes an open- or closed-ended steel casing or pipe pile that is driven and then backfilled with concrete. The advantages

of this type of pile are as follows:

- a. The length can be easily adjusted to match the bearing stratum.
- b. Site-specific modifications, such as an enlarged base or bells, are possible.
- c. Noise and vibration during driving may be reduced.
- d. The internal pile material, such as concrete or grout, will not be affected by handling or driving stresses.

The disadvantages of driven cast in situ piles include these:

- a. Mixing and placement of the concrete or grout may require special equipment and inspection considerations.
- b. The inspection and placement of the reinforcing bar cage may require special considerations or techniques.

- **Self-penetrating (suction) piles.** The placement of suction piles (also called buckets, skirts, suction caissons, anchor piles, or suction anchors) operates on the principle of foundation underpressure (see Figure 34). The suction pile is placed on the sea bottom, and the entrapped water is pumped out to create an underpressure relative to the surrounding water pressure, which causes the pile or caisson foundation to penetrate into the seabed (Senpere and Auvergne 1982). These are commonly called “skirts” in the offshore industry literature if they are attached to gravity-based foundations, especially tension leg platforms. (It should be noted, however, that the offshore literature also refers to “skirt” piles on jacket platforms, which are driven piles instead of suction piles.)

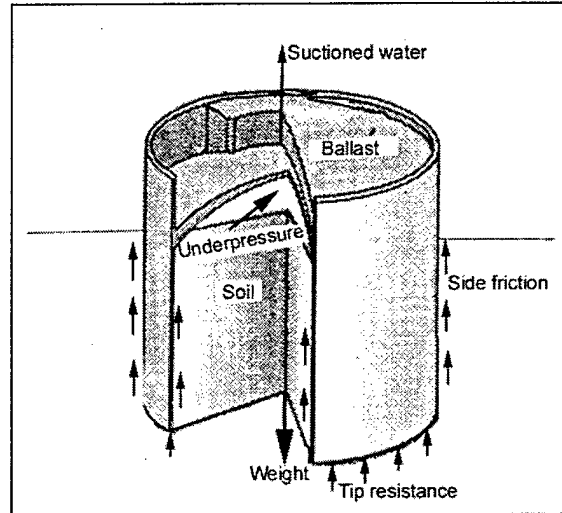


Figure 34. Suction pile foundation (after Andersen and Jostad 1999)

The advantages of self-penetrating (with suction) pile may include one or more of the following:

- Less equipment required for installation.
- Shorter installation time.
- May be fairly easily removed or relocated.

The disadvantages of self-penetrating (with suction) pile include:

- Highly dependent on soil conditions (technology somewhat limited on predicting penetration depth).
- Excessive differential pressure (underpressure) may cause internal soil heave.
- Relatively limited/unproven technology outside the offshore industry.

- **Other types of piles.** Piles may also be modified to increase their performance, design life, and function. One such variation is the spin-fin pile, which screws itself into the subsurface as it is driven. It has been reported that it could provide twice the tensile load resistance as that of similar size conventional piles (ASCE 1992). It should be noted, however, that some of these pile foundation types were designed for onshore applications and are not normally installed in the underwater environment.

Excavated soil (caissons, drilled shafts, and pin piles). These methods include installing the pile, caisson, or drilled shaft during or after excavation of

the soil and/or rock, as illustrated in Figure 35. The advantages of the excavated pile include the following:

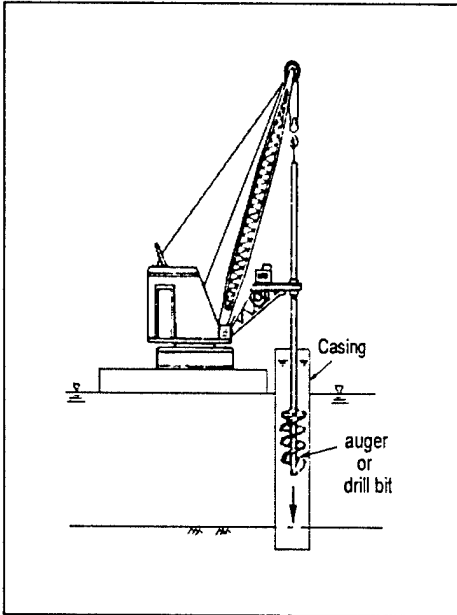


Figure 35. Excavating a cased borehole

- a. Lengths are easily adjusted to match the bearing stratum.
- b. Cuttings supplement or verify site investigation borings.
- c. Very large diameters are possible for extraordinary loads, compared with pile groups.
- d. Suitable for sites that preclude driven foundations, such as cases in which large rocks and boulders are encountered.
- e. The material used to construct the pile is not subject to handling or driving stress.
- f. Very long piles are possible.
- g. Construction noise and vibration are reduced.
- h. Less influence on adjacent structures due to ground heave or disturbance.

Typical disadvantages of excavated piles include:

- a. Concrete and grout require special equipment and inspection considerations.
- b. Placement and inspection of the reinforcing bar cage may require special techniques/considerations.
- c. Cannot be extended above riverbed without casing and special procedures.
- d. Possible end-bearing reduction in sandy stratum.

A brief discussion of several types of (excavated soil) piles follows:

- **Augered cast-in-place.** The augered cast-in-place pile system has been used onshore since the 1940s. Two methods are available. One method consists of drilling a hollow-stem auger to depth and then pumping grout or concrete into the hollow stem as the auger is withdrawn. Reinforcing rods may be inserted into the concrete through the hollow-stem auger (Neate 1989). The excavated soil is brought to the surface as cuttings. For use of this system under water, external casing would be required if the concrete extends above the mudline. The second method uses continuous-flight auger equipment to excavate soil inside a driven

casing. Numerous equipment manufacturers and techniques are available for this popular (onshore) method.

- **Rotary drilled.** Rotary drilling methods are used to create a borehole in soil or rock. Basically, the drill string (drill pipe or casing) is rotated into the subsurface while a vertical load or pressure is applied (Figure 36). The cutting tool rips into the soil or rock, and the cuttings are flushed to the surface by drilling fluid. Several techniques are available to remove the cuttings from the borehole, including positive circulation, reverse circulation, duplex drilling (inner and outer drill strings), percussive rotary, and compressed air. Case histories of in-the-wet foundation installation using rotary drilling methods are more common for rock foundations (Norwesco 1999, Nakayama 1992, Santiago et al. 1986, Burt and Harris 1980, Steinke and Strasser 1978, Chamberlin 1970). However, rotary drilling methods are also used in stiff clays such as those found in the North Sea (Focht and Kraft 1977).

- **Pin piles.** The use of pin piles has increased in recent years, especially in situations with difficult access or restricted vertical clearance. For example, bridge foundation rehabilitation has become a major market for these piles (Pearlman et al. 1997). Pin piles are small-diameter drilled and grouted piles. Their diameters range from 5 to 12 in. (127 to 305 mm), and their axial compression and tension capacities range from 50 to 200 tons (445 to 1780 kN). The most common installation technique is to rotary-drill an open-ended steel pipe into the subsurface. A reinforcing bar cage is then installed inside the casing, and pressurized grout (or tremied grout in rock formations) is pumped into the casing to extend the "bond zone" below the open end of the pile.

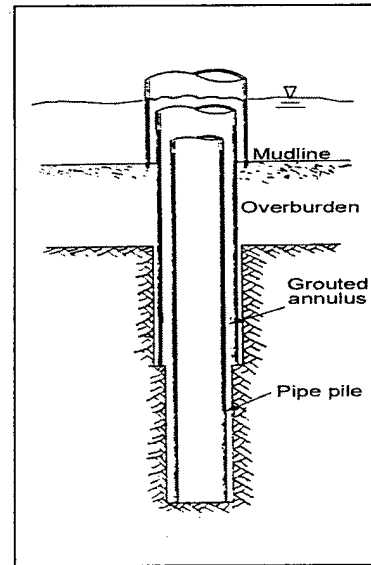


Figure 36. Casing and pile assembly for an offshore drilled shaft foundation (after Steinke and Strasser 1978)

Foundation Selection Criteria and Design Issues

The foundation must be selected, designed, and constructed to achieve acceptable performance during its anticipated lifetime. Selection and design criteria for onshore (land-based) foundations are listed in numerous publications, but published criteria for selection and design of underwater foundations are, in general, somewhat limited. Although the offshore industry has published guidelines for foundation design, installation, behavior, and testing (American Petroleum Institute 1993), the selection guidelines covering all possible types of underwater foundations are not commonly found in the literature. As foundation costs increase and different foundation types become more available to designers and constructors, Reese and Isenhower (2000) suggested that greater emphasis will be placed on selecting the best foundation for a particular application.

The selection procedures for underwater foundations do not differ significantly from those for onshore procedures, as the design requirements for

both types of foundations are similar. The design criteria are dependent upon the appropriate soil/rock properties (that is, shear strength or bearing capacity, settlement, differential settlement, etc.). The major difference in the selection process for underwater foundations stems from unique installation problems and environmental conditions, such as wind, wave, and ice loadings to which many onshore foundations are not subjected. Table 2 addresses some of the general selection considerations.

Underwater foundation selection and design may be somewhat more complicated than for onshore foundations because of the unique loading configurations, installation requirements, and greater uncertainties in site investigation and selection of appropriate soil parameters. However, the offshore industry has greatly advanced geotechnical practices in the last 25 years, and application of those practices will eventually merge into the general foundation selection process. For example, suction piles have become competitive alternatives to driven piles and gravity-based foundations. Risk analysis and partial safety factor design are becoming more commonplace in offshore foundation design and analysis, as is model testing using the centrifuge (Lacasse 1999).

Selection criteria

Engineering and economic analyses are necessary to select the proper foundation system. The traditional choices for selection of onshore foundations are deep foundations or spread footings. Each foundation type is generally associated with given structures. For example, traditional U.S. engineering practice is to specify piles for bridge foundations. It has been reported that 66 percent of U.S. bridge foundations are founded on piles, 25 percent are placed on spread footings, and the remainder are drilled shafts (DiMillio 1999). In contrast, England's highway bridges rely almost extensively on spread footings, even in coal mine subsidence areas where piles could reduce settlement. Likewise, the U.S. building industry almost exclusively uses spread footings, even though some building items, such as doors, windows, and utilities, are much more sensitive to settlement than are piled highway bridges. Thus, foundation selection may be based on industry and local practice, which may not necessarily be the best choice. General selection criteria for underwater foundation systems, which have been compiled from case histories and published information for both onshore and offshore foundations, are discussed below.

Design considerations

Improved-site foundations. An improved-site foundation may be the least complex of all foundations because the foundation itself is not "connected" to the superstructure. An improved-site foundation is usually composed of a leveled surface (excavated or backfilled) overlying either modified or natural soil and/or rock material. Modification of the soil or rock implies that those materials have been physically or chemically altered to be able to transmit the loading imposed

by the superstructure into deeper material without failure due to bearing capacity, settlement, or differential settlement.

Table 2
Underwater Foundation—General Selection Considerations

Site Conditions	Foundation Types	Remarks
<i>Cohesionless soils</i>		
Silica sands		Common in the southern North Sea, Gulf of Mexico, inland U.S. waterways
North		
Fine-grained	IS, GB, P/driven, suction P/bored with casing	
Medium/coarse	IS, GB, P/driven, suction P/bored with casing	
Calcareous sands	P/bored P/driven with grout	Carbonate-containing sands founds in Florida, Hawaii, and the Tropics
<i>Cohesive soils</i>		
Dense silts	P/driven	High penetration resistance without high bearing capacity
Clays		
Soft	IS, P/driven, suction	
Normally consolidated	GB, P/driven, bored, suction	
Stiff to hard	GB, P/driven, bored, suction	
Overconsolidated	GB, P/bored	
<i>Rock</i>		
Weak	GB, P/bored or driven	
Intact	GB, P/bored	
<i>Other site conditions</i>		
Presence of boulders	IS, GB, P/bored	
Corrosive soils	IS	
Contaminated soils	IS, GB, P/driven	Minimize exposure or remediate
Underwater cables	IS, GB	Dredging hazard
Tunnels, buried pipe	IS, GB	Penetration hazard
Potential scour	IS, P/driven	Pipe piles offer less resistance to swift currents; provide scour protection
<i>Design and construction requirements</i>		
Minimal settlement	P/driven	
Maximum lateral loading resistance	P/driven, bored, suction	
Maximum cyclic loading resistance	P/driven, bored	
Maximum uplift resistance	P/driven, suction	
Noise/vibration restrictions	IS, GB, P/bored, suction	
Minimal site QA testing efforts	GB	P/driven with quiet above-water hammer
Legend: IS = improved-site GB = gravity-based P = pinned		

Alternatively, the prepared surface may underlie man-made materials such as geotextiles or bridging mats. The superstructure itself is designed to transmit the imposed loading to the underlying foundation material through its footprint (foundation), which has been designed (sized) to minimize the settlement and movement of the structure. Figure 37 illustrates an improved-site foundation with numerous features, including excavation dredging, backfilling, and mechanical modification by vibratory compaction. Figure 38 illustrates usage of man-made bridging mats and geotextiles combined with dredging excavation and vibratory compaction.

An underwater embankment foundation must be designed for stability and minimal settlement similar to an onshore design. The onshore methods for computing slope stability may be applied with the caveat that spatial variation will be analyzed. Since the underwater variability will likely be greater than for onshore slopes, solutions yielding the single critical shear surface and minimum factor of safety should be expanded to include spatially variable factors of safety. Liedtke and Wright (1999) analyzed this approach using the UTEXAS3 software package.

Dredging, excavation, and (back)fill placement design principles are similar to those used onshore. Namely, unsuitable materials are removed and replaced with more suitable material. Disadvantages of dredging and backfilling include environmental concerns in sensitive marine or riverine environments, contaminated dredged material issues, and possible scarcity of nearby suitable fill materials.

In addition to excavation and replacement, in situ modifications may also be required to increase the soil/rock strength, reduce settlement, or to improve stability. In sands, the primary goal is to increase the relative density. In clays and weak rocks, the primary goal is to increase the undrained shear strength. Three techniques available to achieve those goals are mechanical, hydraulic, and chemical modifications. Mechanical modifications include physical rearrangements of the soil matrix and include modifications by inclusions and/or confinement. Hydraulic modifications include dewatering schemes. Chemical modifications include grouting and soil mixing schemes (Hausmann 1990).

Mechanical modifications using reinforcing mats, steel mats, prefab rock blankets, or geotextile fabrics have the effect of creating dense layers with improved strength characteristics, as well as serving as filters to minimize erosion or scour. The foundation bearing capacity and settlement design then becomes a multiple-layer subgrade analysis with stiffer and more competent layers overlying softer strata. For example, Abbot and George (1989) determined that steel reinforcement meshes overlaid with granular layers would reduce potential settlement by 66 percent and increase shear strength by a factor of 800 as compared to the unimproved site.

Mechanical modification using vibrocompaction is sometimes referred to as vibroflotation if water jetting accompanies the vibration. Vibrocompaction mechanically rearranges soil particles into a denser configuration by the use of vibrating shafts and is used exclusively for cohesionless soils. Vibrating shaft

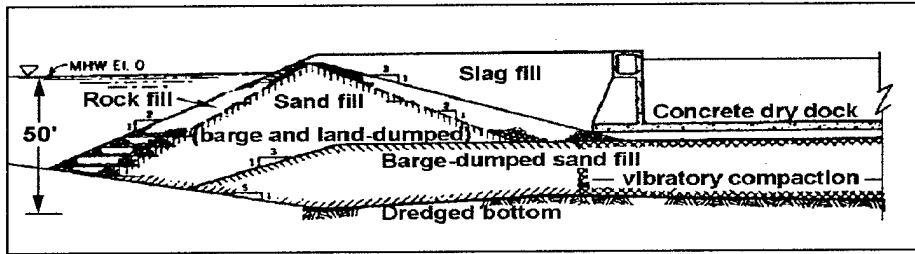


Figure 37. Improved-site foundation under Bethlehem Steel's graving dock near Baltimore (after Millard and Hassani 1971)

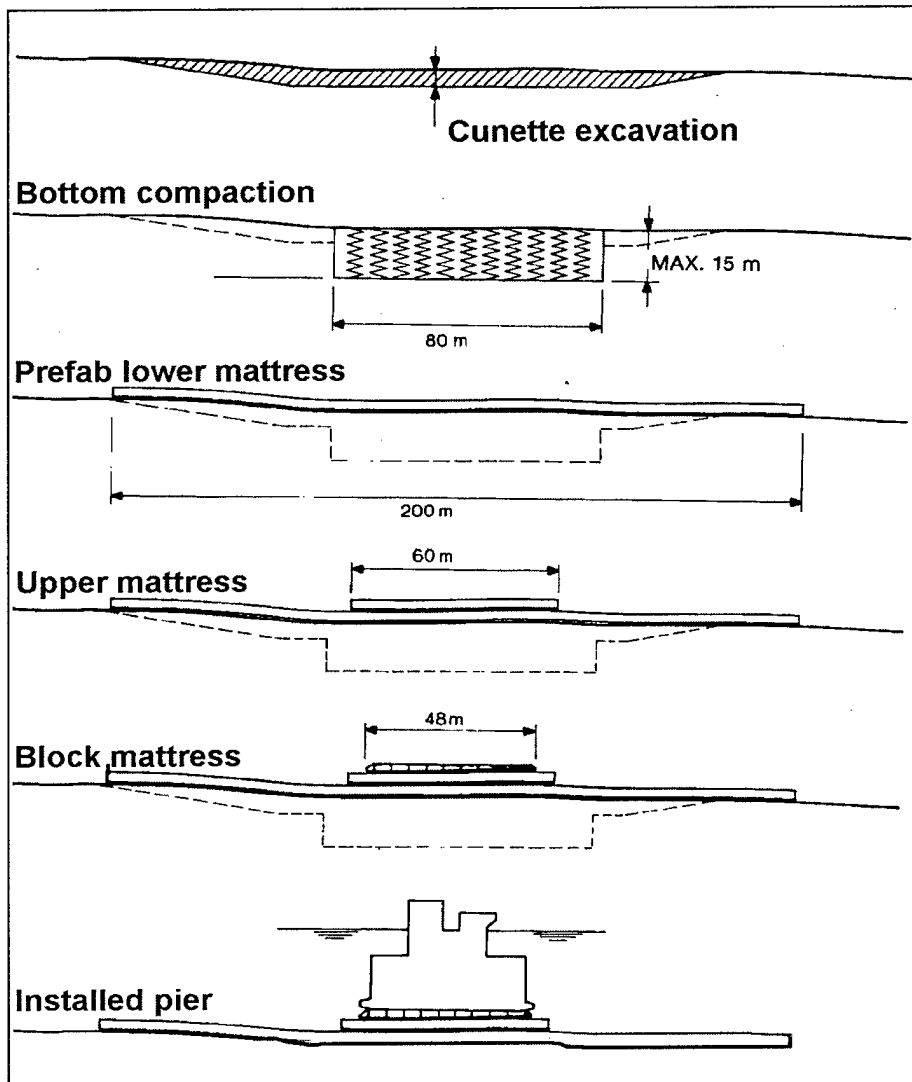


Figure 38. Eastern Scheldt improved site foundation sequence for gravity-based piers (after DOSBOUW 1987)

equipment may be specially designed or may simply be a vibratory hammer attached to a pipe pile. The effectiveness of vibrocompaction in silty sands is marginal to good but in silts and clays is poor (Hayward Baker 1999a). The design process for vibrocompaction consists of first calculating the predicted settlement (knowing soil gradation and relative density) and then designing sufficient densification using vibrocompaction to reduce the expected settlement to a desired value. The final density is determined by acceptance testing using standard penetration, cone penetrometer, pressuremeter, or actual load tests. Several case histories document the use of vibrocompaction techniques to densify cohesionless soils. Tate (1961) reported that vibroflotation performed on drained sands achieved a relative density increase up to 90 percent from the original 50 percent; Millard and Hassani (1971) disclosed that use of a vibratory hammer/pipe pile to densify a cohesionless soil increased the bearing capacity by 100 percent and reduced design settlement by 50 percent; and Jonker (1987) cited similar results using underwater vibrocompaction of the cohesionless materials in the Eastern Scheldt project.

Hydraulic modification to consolidate cohesive soils involves drainage techniques. Both vertical and horizontal drains have been successfully used underwater. The first documented underwater installation of vertical drainage wicks in North America was performed during site improvement prior to construction of the Pascagoula (Mississippi) Naval Base. More than 4,000 wicks were placed underwater to consolidate the underlying soft bottom silts at the site of a new berthing pier. A 20-ft (6-m) sand surcharge was used to cause the consolidation (Munn 1989). Disadvantages of wick drains include the need for specialized equipment, the requirement to provide consolidation surcharge, and the time required for consolidation to achieve significant gains in soil strength.

Horizontal drainage to dewater silt layers at underwater depths of 64 ft (20 m) has been accomplished using a vacuum pumping system, as illustrated in Figure 39. The purpose of dewatering was to allow larger volumes of dredged

material to be placed in an underwater disposal area. Collector pipes were installed as successive silt layers were added to the disposal area.

Storage capacity of the 986- by 1,300- by 16-ft (300- by 400- by 5-m) underwater disposal area was increased about 20 percent by dewatering the silt. Density after dewatering was increased by about 5 percent (Aerts, Devlieger, and Vandycke 1999).

Chemical modification techniques have been used for relatively shallow soft soil seabeds for bottom-founded offshore oil

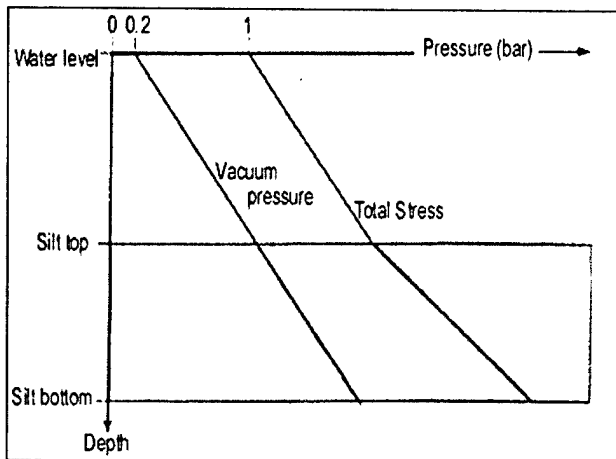


Figure 39. Suction dewatering in an underwater silt layer (after Aerts, Devlieger, and Vandycke 1999)

structures in the Arctic and for harbor areas of Japan. In the Alaskan Beaufort Sea, man-made islands have been constructed in the relatively shallow water depths of 10 ft (3 m) to support drilling platforms. Halebsky and Wetmore (1986) discussed several methods of soil modification and concluded that, of the methods proposed (dredging/backfilling, soil wick drains, soil freezing, and deep cement mixing), deep cement mixing offered an economical advantage. The dredging/backfilling and soil wick drains methods were assumed to increase the original soil strength by a factor of 2 whereas the deep cement mixing method (in situ soil strengthening using cement, or DCM) was found to increase the original soil strength by a factor of 70. In many locations, especially in Arctic regions, soils with shear strengths as low as 500 psf (24 kPa) are found. Since exploratory oil-drilling structures require soil shear strengths of 1,000 to 2,500 psf (50 to 20 kPa) and permanent structures require greater shear strengths, increasing the soil shear strengths by a factor of 10 to 20 would make most weak soils acceptable.

The major disadvantages of the DCM technology include the specialized equipment that is required and the required time for curing. This technology for underwater applications is more commonly used in Japan than in the United States. Japanese DCM barges operate in water depths to 230 ft (70 m) and modify the soil to depths of 165 ft (51 m) beneath the seafloor. Typical curing times to achieve maximum design strengths are 2 to 3 months.

Gravity-based foundations. For gravity-based foundations, the superstructure is “connected” to the foundation (footprint), and the foundation is designed to resist the imposed compression, tension, overturning, lateral, and dynamic forces. Design should include the following considerations (as illustrated in Figure 40):

- a. Stability against overturning, bearing capacity failure, sliding, or combinations thereof.
- b. Static deformation (settlement and differential settlement).
- c. Dynamic loading and response.
- d. Hydraulic instability from scour or wave pressure piping.
- e. Construction/installation effects, such as excessive pore-pressure buildup during base installation.

In general, onshore shallow foundation design procedures are used (American Petroleum Institute 1993). The bearing capacity of the soil is calculated using the undrained strength parameters ($\phi = 0$) for clays or the drained strength parameters for sands. The shear strength of the soil/rock is converted into resisting values for the imposed loading conditions by the selection of appropriate bearing capacity factor(s). Applicable soil parameters are inserted into standardized equations, and the failure load is calculated.

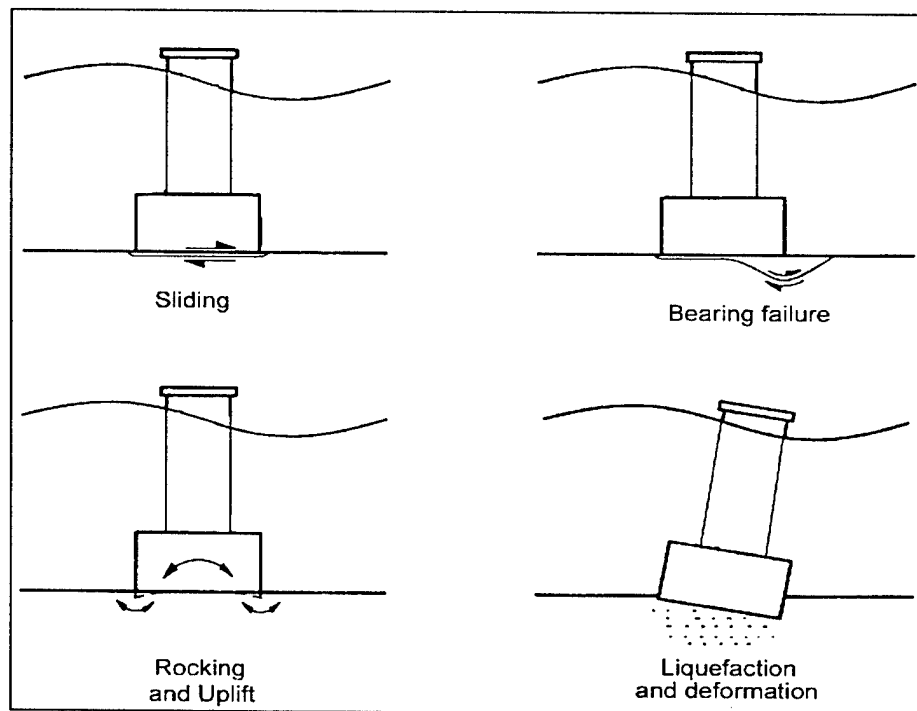


Figure 40. Possible failure modes of a gravity-based foundation

Dividing the failure load by the effective foundation area yields the ultimate bearing capacity (q_u). The ultimate bearing capacity is then divided by the required (or chosen) safety factor.

The American Petroleum Institute (API 1989) recommends (with caveats) a safety factor of 2 for underwater foundations, while the Department of the Army (1983) recommends a safety factor from 1.5 to 3 for nonhydraulic structures, depending upon the values of the dead and live loads. Sliding stability calculations take horizontal and inclined loads into account. The maximum horizontal load at soil failure is calculated for either a drained or an undrained condition. API's recommended safety factor is 1.5.

A more comprehensive approach to design is to couple the limit states design (LSD) concept with probabilistic assessment. This concept considers two limit states: ultimate limit states (ULS) and serviceability limit states (SLS). ULS involves soil strength, ultimate bearing capacity, overturning, sliding, and other structural safety conditions. The ULS methodology incorporates the load and resistance factor design (LRFD) concept, which uses partial factors accounting for uncertainties and variability. The SLS methodology involves conditions such as differential settlements from construction and service loads. Becker (1996) provides an overview of the LSD process for gravity-based foundation design.

Layered soils require additional considerations to prevent "punch through" (bearing capacity failure). The majority of jack-up drilling rig accidents are caused by foundation problems, and the majority of foundation problems are

caused by punch-through on layered soils (Sharples, Bennett, and Trickey 1989). Senner and Cathie (1993) discuss methods to design and assess likelihood of bearing capacity failures on layered soils, including (a) strong clay overlying weak clay, (b) sand overlying clay, (c) strong soil overlying weak soil, and (d) thin weak layers.

Calcareous (carbonate) sands require special attention. They may exhibit high friction angles in laboratory testing, which leads to overestimating actual bearing capacity. However, the load-carrying capacity of carbonate sands may be somewhat lower because of their highly compressible behavior (Dutt and Ingram 1988, LeTirant and Nauroy 1994).

Static deformation considerations include both short- and long-term effects. The compression index and the coefficient of consolidation are obtained from laboratory (or other suitable type test) consolidation test data. The vertical stress distribution for the before- and after-loading conditions can be calculated using elastic theory. Using these criteria, the total and differential settlements can be calculated and compared with the allowable displacements.

Considerations of dynamic behavior include cyclic loading from wind, current, waves, ice, and earthquakes. Soil-structure interactions must be examined for each anticipated loading situation. Failure during cyclic loading may be associated with large horizontal or rotational cyclic displacements, large settlements, or combinations thereof. The undrained bearing capacity of certain clayey soils may be significantly lower during cyclic loading than monotonic loading. To analyze the static bearing capacity after cyclic loading, the postcyclic static shear strength must be known (Andersen, Kleven, and Heien 1988). Andersen and Lauritzsen (1988) describe procedures for determining the cyclic and postcyclic bearing capacity of gravity-based foundations on undrained clay. Typical characteristic cyclic loads imposed on a North Sea gravity-based foundation are these: (a) a horizontal wave load of 67,443 tons (600 MN), (b) a wave moment of 3,417,096 ton-ft (100,000 MNm), and (c) a vertical wave load of 44,962 tons (400 MN). These cyclic loads are based on the typical design storm with a 100-year return period (Andersen, Kleven, and Heien 1988).

Scour skirts, riprap, or other scour protection methods are designed to prevent soil erosion and undercutting. Excessive hydraulic gradients during and after foundation installation may cause piping and excessive disturbance to the supporting soil.

Designs of gravity-based foundations sited on rock generally follow the same approach as for soils, but use slightly different methods. The failure methods are the same as for soils, but the mechanisms are different. The geotechnical parameters are also different. Rock strength classification is typically based on either the RMR system (Bieniawski 1974) or the Q system, also called the NGI system (Barton, Lien, and Lunde 1974). Appropriate rock strength parameters are input, and allowable resistance values for bearing and sliding are calculated using safety factors or probabilistic methods. Becker et al. (1998) discussed design methods for underwater bridge piers founded on weak rock.

Pinned foundations. Pinned foundations are typically employed to provide end-bearing and/or frictional resistance to imposed compression, tension, lateral, and dynamic loads. Pinned or pile foundations are installed into the subsurface soil and/or rock either by displacement or excavation. Impact-driven piles displace (dislocate) the soil. Drilled caissons or shafts require borehole excavation. Most of the offshore structures use pinned foundations consisting of driven piles. Lacasse (1999) reported that there are an estimated 6,000 offshore piled structures around the world.

As a very general rule, bored piles are selected where there is a relatively shallow bedrock stratum to develop end-bearing resistance, whereas driven piles are selected for soils in which a high friction resistance can be used. However, this dividing line is not distinct, and the pile type selection is usually governed by installation economics and environmental conditions such as noise and vibration (Thornton 1992). The most common offshore foundation type is a driven tubular open-ended (pipe) pile designed for axial loading with minimum lateral loading in granular and cohesive soils (ARGEMA 1992). An offshore pipe pile typically has a diameter of about 7 ft (2.1 m), a wall thickness of about 2 in. (5 cm), and penetration length of 300 to 400 ft (90 to 120 m). The pile's size and length are commonly determined by the required bending moment and by the penetration depth to develop the necessary resistance to compressive and tensile loads. As a rule of thumb for design purposes, Gerwick (1971) suggested that the maximum compressive load was approximately twice that of the maximum tensile load. In the event that excessive soil resistance to driving (refusal) is reached prior to the designed tip penetration, several techniques are available to achieve design penetration and include combinations of driving and drilling methods.

After the type of pinned foundation has been selected, the designer must then determine the number, length, and size of the piles or drilled shafts. Several design guides for piles and shafts are available, including API RP2A (1993); Federal Highway Administration guides for bridge foundations, piles, and drilled shafts (Hannigan et al. 1997; Armour and Groneck 1998; Raushe, Goble, and Moses 1997; Bruce and Juran 1997; Baker et al. 1993; Barksdale and Bachus 1983; Lam and Martin 1986; O'Neill 1988; and Reese 1984); U.S. Army Corps of Engineers 1991 and 1994 (Engineer Manuals 1110-2-2906 and 1110-1-2908); American Society of Civil Engineers (1981, 1984, and 1993); Construction Industry Research and Information Association (1993); and other publications (Whitaker 1976, ARGEMA 1992).

The offshore industry has used the recommended empirical design methods of the American Petroleum Institute, since its API RP2A was first published in 1969, primarily for Gulf of Mexico fixed platforms founded on open-ended steel pipe piles. Several iterations over the years have produced the latest design recommendations (API 1993). The design guidelines are based on extensive research and load tests, extrapolated to the actual loads imposed on offshore structures. It is interesting to note that although the largest pile capacity in the API database is about 4,000 kips (18 MN), typical axial capacities (loads) for offshore piles range from 10,000 to 15,000 kips (44 to 67 MN) per pile. For compliant towers, the pile capacities are about 30,000 kips (133 MN) per pile.

Although the design guidelines have been vigorously extrapolated, the field results have remained acceptable (Pelletier, Murff, and Young 1993).

API's design format uses both the Working Stress Design and the Load and Resistance Factor Design methods. In deference to the European Community process of developing international offshore platform design standards through the International Standards Organization (which has a reliability-based format), the LRFD method will likely predominate (Pelletier, Murff, and Young 1993; Toolan and Horsnell 1993). The LRFD method includes partial safety factors (reduction factors for soil parameters and multiplication factors for loads) instead of lumped safety factors.

Other international offshore pile design codes are regulated by Det Norske Veritas (DNV 1989) and the Department of Energy, U.K. (1986). Both these design codes and the API recommendations include procedures for designing in cohesive and cohesionless siliceous soils. The ARGEMA (1992) and CLAROM (1994) design guides provide specific guidance for calcareous (carbonate) soils.

Recommended design parameters for cohesionless siliceous soil are given in API (1989, 1993) for both driven and drilled/grouted piles. The limiting unit end-bearing values and limiting skin friction values range from 40 to 250 ksf (2 to 12 MPa) and 1 to 2.4 ksf (50 to 115 kPa), respectively, as the soil density increases from very loose to very dense. In calcareous sands (containing calcium carbonate), driven piles may have substantially lower design strength parameters than drilled and grouted piles.

The design methodology for both driven and drilled piles is similar. It consists of comparing the applied loads to the soil's internal capacity and adjusting these values using applicable safety limits. The ultimate load-carrying capacity is the sum of the pile end-bearing capacity (soil's resistance to bearing) and the pile skin friction capacity (soil's frictional resistance):

$$Q = Q_p + Q_f$$

where

Q = load-carrying capacity of the pile

Q_p = end-bearing load

Q_f = friction load

and $Q_p = qA_p$

where

q = soil's end-bearing capacity

A_p = pile end area

and $Q_f = fA_s$

where

f = soil's skin friction capacity

A_s = side surface area

The required diameter and length (penetration depth) of the pile are calculated using A_p and A_s . For given soil values of q and f , the ultimate load-carrying capacity Q increases as pile diameter or penetration depth increases. Ultimate-capacity design uncertainties are usually related to installation uncertainties (plugging and drivability) and soil-capacity uncertainties (q and f values).

Alternatives—for situations in which design objectives cannot be obtained during the installation of the pile—should be identified and defined in the design process (API 1989). For example, in the event sufficient penetration cannot be achieved, removal of all or part of the soil plug is an option that can be used to permit driving of the pile to the design depth. Care must also be taken when basing pile design on load test correlations using soil properties determined from both in situ tests and the laboratory test results. For example, if the in situ test results yield shear strengths that are higher than those obtained from laboratory samples and the load test correlations were made based upon the laboratory data, using the higher values of shear strengths from in situ tests may lead to an unsafe design (Reese and Isenhower 2000).

Besides designing for ultimate-capacity loading, the pile material must be able to withstand handling and transportation stresses. Wall thicknesses may be governed by the loads developed during installation procedures, especially if the contractor opts to provide single-piece installation instead of making field connections (Will et al. 1999). Column buckling due to unsupported lifting from the horizontal to a vertical orientation or the dead weight of an underwater hammer clamped to the top of a tilted pile undergoing self-penetration is an important design consideration (Doyle 1999). Steel fatigue due to hammer driving is another material design consideration, especially when a large number of blow counts are anticipated in the soil resistance to impact driving analysis.

Varying wall thicknesses along very long offshore pipe pile lengths is a commonly noted design consideration, as some piles extend to about 400 ft (122 m) in length. Typically, the pile walls are thicker at the mudline due to higher axial and bending stresses in that area (Davies and Srivareerat 1999). Cunningham and Naughton (1977) discussed design aspects for a 1,255-ft (382-m)-long, 4-ft (1.2-m)-diameter, 50-ksi (345-MPa) yield stress pipe pile driven to 375 ft (114 m) penetration. Wall thickness at the mudline was 3 in. (7.6 cm), decreasing to 1 in. (2.5 cm) above the mudline depth.

Pile-driving studies are required to determine the soil's resistance to impact driving (SRD) or vibratory driving (SRV). For example, if the pile cannot be driven to the design penetration depth, the pile's designed ultimate capacity may not be achieved. SRD or SRV must be studied as part of the design process since

these values determine the likelihood of achieving a successful pile installation during the construction phase.

Two aspects of the pile-driving study that should be addressed are (a) selecting the proper pile and hammer and (b) establishing criteria to define a successful installation. Hammer selection may be the most important aspect of pile installation (ASCE 1993). Numerous resources are available for pile-driving equipment selection (Department of the Army 1982). The wave equation analysis provides guidance for items (a) and (b) above and is discussed in detail in Chapter 5. Proof load testing prior to finalizing the design specifications also provides valuable guidance in establishing criteria defining a successful installation. Load testing is also discussed in Chapter 5 of this report.

Self-weight penetration (prior to hammering) as the pile is lowered into the mudline is a design consideration. Self-weight penetration may be calculated by assuming that the penetration resistance is provided by outer circumference skin friction, end bearing at the pile base, and inside skin friction provided by the shoe. The buoyant weight of the pile and attached lifting tools provides the penetration (driving) force. Doyle (1999) noted that unconsolidated-undrained remolded soil strength values were used for skin friction, and undisturbed soil strength values were used for end-bearing resistance in self-weight pile penetration calculations at clay sites in the Gulf of Mexico. At those sites, self-weight penetration was roughly 25 percent of the total pile lengths.

Designing drilled shafts socketed into bedrock generally relies on shaft resistance instead of end bearing, since shaft resistance is fully mobilized at much lower deformation as compared with end-bearing resistance (Becker et al. 1998). For example, the design of the underwater approach piers for the Confederation Bridge was initially based on unit shaft resistance values of 7.5 to 14.1 ksf (360 to 680 kPa) developing in the mudstone, siltstone, and sandstone layers. Drilled shaft lengths averaging 44 ft (13.5 m) with 6.5-ft (2-m) diameter were then calculated based on factored design loads up to 6,745 tons (60 MN) vertical, 73,953 ft-tons (200 MN-m) bending, and 2,135 tons (19 MN) horizontal. Design shaft lengths were later reduced to 25 ft (7.5 m) based on actual (onshore) load tests that indicated much higher shaft resistance values in the rock layers.

Full-scale load tests of drilled shafts using the Osterberg cell method have typically shown that the maximum applied test load is much larger than the design load, including the factor of safety. According to Osterberg (1998), relatively few designers have taken advantage of these results by performing economical redesign efforts. One extreme overdesign example revealed that when a test shaft was loaded to 3,000 tons (26.5 MN) with no sign of failure, the design firm was pleased that the design load of 500 tons (4.4 MN) was validated. No efforts were made to redesign the shaft to a more economical, smaller size.

The effects of scour around pinned foundations requires reducing the effective shaft length to account for reduction in side shear (friction resistance). For example, drilled shaft designs for the Ohio River Bridge (Osterberg 1998) took into account only the bottom 19 ft (6 m) of shale. The load capacity of the overlying 63 ft (19 m) of sand and gravel was not considered because of the

possibility of deep scour. Full-scale load testing (using the Osterberg cell) was tailored to reduce the overburden's load-carrying contribution.

The design criteria for a suction pile is similar to the gravity-based and pinned foundation design methodologies. For the suction pile, the penetration resistance is a function of the soil's resistance to bearing and friction and is calculated as the sum of the shear along the walls and the bearing capacity at the tip. The penetration analysis includes calculation of the underpressure (the difference between the hydrostatic water pressure outside the pile and the water pressure inside the pile cavity). The safety margin against soil heave inside the cavity must be determined to calculate the allowable underpressure (Andersen and Jostad 1999). The calculation of self-weight penetration of the suction pile is a critical design aspect that must be addressed due to its effect on the pile cavity volume.

The bearing capacity and holding (tension) capacity of suction piles are calculated by limit equilibrium methods. The critical failure surface depends on the combination of vertical, horizontal, and moment loads acting on the pile. Andersen and Jostad (1999) provide comments on design concepts and procedures for suction pile (skirted foundations and anchor) design. The following design aspects need to be analyzed for suction piles:

- Penetration
 - resistance
 - necessary and allowable underpressure
 - soil heave inside cavity
 - maximum penetration depth
 - factor of safety
 - removal analysis if needed
- Capacity
 - limiting equilibrium modeling
 - shear strengths
 - soil crack effects
- Displacements
 - consolidation
 - cyclic and permanent displacements due to cyclic loads
- Soil spring stiffnesses (dynamic analysis)
- Soil-structure interaction

Analysis and verification testing

Design analysis is conducted during the design phase, and may include numerical and physical modeling. Numerical modeling techniques (such as finite element analysis) and physical modeling techniques (such as model tests) can be used during this phase.

Model tests are geotechnical investigative tools that best document the failure mechanism, deformation patterns, reliability of numerical models, and soundness of the calculated design (Lacasse 1999). Model tests can range from

full scale to small-scale prototypes and may include the use of centrifuge testing. Figure 41 illustrates model scaling relationships for centrifuge tests.

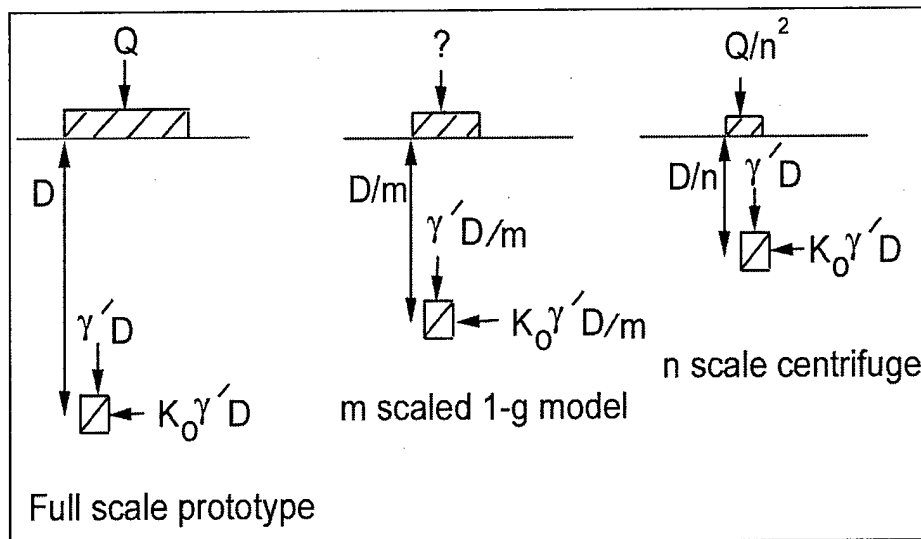


Figure 41. Scaling relationships for 1-g and centrifuge models (after Murff 1996)

When properly designed and conducted, centrifuge testing offers significant advantages compared with 1-g model testing. Centrifuge testing of undrained, partially, or fully drained soils with simplified profiles offers an advantage. The disadvantages include the use of miniature instrumentation, the effects of time scale, and relatively high costs. Murff (1996) presents a discussion of centrifuge testing aspects related to offshore foundation design and analysis that was supplemented with several case histories.

Published examples of centrifuge testing for offshore foundations are not as common as those for onshore applications, perhaps due to the proprietary nature of some tests. Nicola and Randolph (1999) discussed model pile test results compared with current guidelines for offshore pile design. Hu, Randolph, and Watson (1999) discussed centrifuge testing and finite element analysis of offshore skirted gravity-based foundations. Becker et al. (1998) discussed centrifuge testing and finite element analysis of an underwater gravity-based foundation for a bridge pier.

Quality Control and Assurance Issues

Quality control and quality assurance (QA/QC) are essential in engineering design and construction. Quality control and assurance elements are especially important for in-the-wet construction projects where the foundations are prepared, installed, and tested in an underwater remote location, often from a floating platform on the river or marine surface. Visual inspection of the

foundation before, during, and after construction is generally not possible, as compared with the onshore process where the aboveground portion of the foundation is easily viewed and accessible. Specialized techniques and equipment are needed to install quality foundations underwater, and adequate QA/QC procedures must be specified and followed to achieve and ensure successful installation.

Quality control and quality assurance are not the same functions, and are more than an inspection process. Quality control refers to setting up and maintaining a system that provides a means to *control and measure* specified characteristics (of the underwater foundations). Quality assurance *verifies* that the quality control system is performing adequately. Quality control and assurance operate hand-in-hand to enable the underwater foundation to be constructed in strict accordance with the engineering design and specifications by monitoring, recording, and certifying the entire process. Adequate QA/QC is the collective responsibility of all parties concerned with designing, specifying, manufacturing, purchasing, and installing the various project components (Batten 1987). Although the QA/QC system applies to the planning, conceptual design, and engineering design phases of a project, its most common and better defined application is in project construction. Constructing the underwater foundation according to the design plans and specifications requires a high degree of QA/QC due to the complexities and variables involved in contractual construction projects. Preventing the introduction of defective materials or poor workmanship in a construction project before these problems occur constitutes an effective quality control and inspection scheme (API 1989).

Engineering design phase

Design quality is the responsibility of everyone involved in the development of a project (Department of the Army 1994). Several QA steps are involved in the design of Corps projects before plans and specifications are inked, as outlined in Engineer Regulation (ER) 1110-2-1150. Appendix F of the ER is an internal management control checklist to ensure that all engineering functions are conducted in a cost-efficient manner. Adequate coordination, communication, and lessons-learned applications are stressed in order to achieve a high standard of technical quality during the design phase.

Appendix E of ER 1110-2-1150 includes instructions for writing a "report on engineering considerations and instructions for field personnel" who will be responsible for the oversight of the contractors' quality control program. It is stressed that this report should be distributed to appropriate construction field personnel for their suggestions and input prior to completion of the design phase. Field personnel may have a more intimate knowledge of site conditions and potential site-specific design problems than do the engineering design personnel. Hence, the field comments and suggestions should be incorporated early on, especially during the design phase.

The Corps of Engineers has published engineer manuals, technical manuals, and other guidance to ensure consistent criteria and requirements for developing

design and engineering documents for civil and military projects. ER 1110-345-700 covers military and hazardous waste projects. Other regulations are applicable and will not be listed here.

A comprehensive listing of U.S. government standards, design guides, engineer manuals, national standards, building codes, and trade organization standards can be found at <http://www.hnd.usace.army.mil>.

Specifications and contract document phase

Specifications delineate the project requirements and establish exactly how the designer intends the project drawings to be applied. Specifications form a primary tool for setting up the QA/QC system and must provide enough detail for the construction to proceed in a timely, efficient manner. Specifications also provide a basis for estimating the project costs, which impacts contractor selection during the bidding process.

ER 1110-1-8155 (Department of the Army 1998) prescribes specifications policy for the Corps. The Corps of Engineers Guide Specifications (CEGS) are a system of master guide specifications that define the qualitative requirements for products, materials, and workmanship for repetitive work features occurring on Corps construction projects. The Internet site <http://www.hnd.usace.army.mil> contains the CEGS library and additional specification database links. Although usage of the CEGS system is not mandatory within the Corps, the system is updated and maintained to include lessons learned during design and construction, and thus constitutes a valuable source for technical specifications.

No CEGS documents currently exist for underwater foundation design and construction. Various elements of land-based foundation construction that may be applicable to underwater foundation construction, which are covered in the February 1999 CEGS database, are listed follow:

- 01270 Measurement and Payment
- 01451 Contractor Quality Control
- 02210 Subsurface Drilling, Sampling, and Testing
- 02217 Foundation Preparation
- 02251 Foundation Drilling and Grouting
- 02300 Earthwork
- 02315 Excavation, Filling, and Backfilling for Buildings
- 02316 Excavation, Filling, and Backfilling for Utilities
- 02375 Geogrid Soil Reinforcement
- 02378 Geotextiles Used as Filters
- 02453 Prestressed Concrete Piles for Civil Works
- 02454 Precast Concrete Piling
- 02455 Cast-In-Place Concrete Piles, Steel Casing
- 02456 Steel H-Piles
- 02457 Round Timber Piles
- 02458 Prestressed Concrete Piling
- 02459 Piling: Composite, Wood, and Cast-In-Place Concrete

02461 Round Timber Piles for Hydraulic Structures
02463 Steel H-Piles for Civil Works
02464 Metal Sheet Piling
02465 Auger-Placed Concrete Piles
02466 Drilled Foundation Caissons
02722 Graded, Crushed Aggregate Base

Corps Divisions and Districts are given the flexibility to add non-CEGS specifications within a specific construction project package, but inclusion of those specifications within the CEGS system requires proper submission and Headquarters approval. The specification sections must be properly formatted in accordance with ER 1110-1-8155. The CEGS layout for a construction specification (Construction Specifications Institute 1996) follows:

Part 1 General

- 1.1 References
- 1.2 Measurement and payment
- 1.3 System description
 - 1.3.1 Design requirements
 - 1.3.2 Performance requirements
- 1.4 Submittals
- 1.5 Qualifications
- 1.6 Regulatory requirements
- 1.7 Field examples
- 1.8 Mock-ups
- 1.9 Pre-installation conference
- 1.10 Delivery, storage, and handling
- 1.11 Project/site conditions
 - 1.11.1 Environmental requirements
 - 1.11.2 Existing conditions
 - 1.11.3 Field measurement
- 1.12 Sequencing and scheduling
- 1.13 Warranty
- 1.14 Maintenance
 - 1.14.1 Maintenance service
 - 1.14.2 Extra materials

Part 2 Products

- 2.1 Materials
- 2.2 Manufactured units
- 2.3 Equipment
- 2.4 Components
- 2.5 Accessories
- 2.6 Mixes
- 2.7 Fabrication
 - 2.7.1 Shop assembly
 - 2.7.2 Shop/factory finishing
 - 2.7.3 Tolerances
- 2.8 Tests, inspections, and verifications

Part 3 Execution

- 3.1 Examination
- 3.2 Preparation
 - 3.2.1 Protection
 - 3.2.2 Surface preparation
- 3.3 Erection
- 3.4 Installation
- 3.5 Application
 - 3.5.1 Special techniques
 - 3.5.2 Interface with other products
 - 3.5.3 Tolerances
- 3.6 Field quality control
 - 3.6.1 Tests
 - 3.6.2 Inspection
 - 3.6.3 Manufacturers field service
- 3.7 Adjusting/Cleaning
- 3.8 Demonstration
- 3.9 Protection
- 3.10 Schedules

Once the project specifications are sufficiently detailed and written, they are reviewed and approved for inclusion in the project contract documents. During the bidding process and after contract award, conflicts may arise because of problems with the specifications. Based upon one legal firm's breakdown of construction specification conflicts, the following specification problem areas are listed in order of occurrence (Nielsen and Nielsen 1981):

- ***"Or equal" clauses:*** These statements lead to the highest occurrence of specification problems (claims and litigation) because of disputes over what is "equal" in a particular situation. This situation occurs mainly in public works contracts in which the specifier wants a particular brand name product but also must comply with Federal contract law to not limit competition.
- ***Constructibility*** (defective specifications and tolerances): Constructibility problems arise when the item cannot be built, installed, or will not perform as specified. Holding the contractor to tighter tolerances than normal industry standards may also lead to conflict.
- ***Ambiguities*** (phrasing and typographical errors): These problems refer to interpretation issues.
- ***Conflicts between plans and specifications*** (and typographical errors): These conflicts arise from lack of adequate QA/QC during the engineering design phase.
- ***Inaccurate technical data:*** This category includes errors and omissions such as specifying subsurface information that does not match the actual conditions (differing site conditions), specifying inadequate quality control standards, and specifying owner-furnished plant and equipment that is not suitable.

- **Product performance deficiencies:** These may cause problems when new and relatively untried products are specified. This type of problem has direct applicability to underwater foundations contracted by the Corps because the Corps does not have an extensive experience database gained from in-the-wet construction projects. Special emphasis must be given to prebidding demonstrations of products and equipment and postaward presence of knowledgeable field representatives responsible for implementation of new products, procedures, or equipment to be used for in-the-wet projects.

- **Inspection and excessive testing:** Overzealous inspection and testing may also cause specification problems and contract conflicts.

Recent Corps of Engineers underwater foundation contracts

Pittsburgh District. Plans and specifications have been finalized and construction has begun for an innovative Corps in-the-wet float-in construction project. A new gated dam will be constructed at Pittsburgh District's existing Braddock Locks and Dam 2 on the Monongahela River in Pennsylvania (U.S. Army Corps of Engineers 1998). The underwater foundation system consists of a dredged area with sheet-pile cutoff walls in the riverbed alluvium and concrete drilled shafts socketed into bedrock. The precast concrete dam elements will be floated in and ballasted upon and anchored to the drilled shafts. The foundation specifications were custom-written for this unique project and contain numerous non-CEGS specifications. Specifications relating directly to the underwater foundation system include:

- 01025 Measurement and Payment
- 01450 Survey and Alignment Control
- 01451 Contractor Quality Control
- 02012 Exploratory Drilling
- 02013 Exploratory Pile Driving
- 02164 Rock Anchors
- 02273 Scour Protection
- 02314 Seepage Cutoff Walls
- 02316 Steel Pipe Piles
- 02315 Steel H-Piles
- 02383 Drilled Shafts
- 02411 Metal Sheet Piling
- 02482 Dredging
- 02722 Graded Gravel Base
- 03900 Concrete: Drilled Shafts
- 13530 Instrumentation

The predominant work feature for the underwater foundation system is drilled shafts, which provide the deep-foundation support for the concrete float-in elements and function as anchor piles for positioning the float-in elements. The drilled shafts (similar to caissons or piers) consist of steel pipe piles (casing) driven with a vibratory hammer to bedrock. Upon completion of the driving of the casing, rotary drilling was conducted through the pile beyond its open bottom

end and into bedrock (without casing). After the borehole had been advanced to the specified depth in the bedrock, steel reinforcement cages (illustrated in Figure 42) with attached instrumentation tubes (for sonic testing) were to be lowered into the casing and through the bedrock to the bottom of the borehole. The borehole and casing were then backfilled with tremie concrete. Foundation details are presented in Figure 43.

Section 01451, Contractor Quality Control, established the baseline requirements for the quality control system to be implemented by the successful bidder. Some of the novel features of this specification section include:

- a. Requirement for completion of the "Construction Quality Management for Contractors" training course.
- b. Requirement for project management staff to have specialized experience:
 - (1) Project Engineer: minimum 3 years marine construction experience
 - (2) Concrete Superintendent: experience in mass concrete operations
 - (3) Marine Superintendent: minimum 15 years marine project experience
 - (4) Diving Superintendent: minimum 5 years diving supervision
 - (5) Chief Surveyor: minimum 5 years marine surveying experience
 - (6) Project Safety Engineer: minimum 3 years marine construction experience

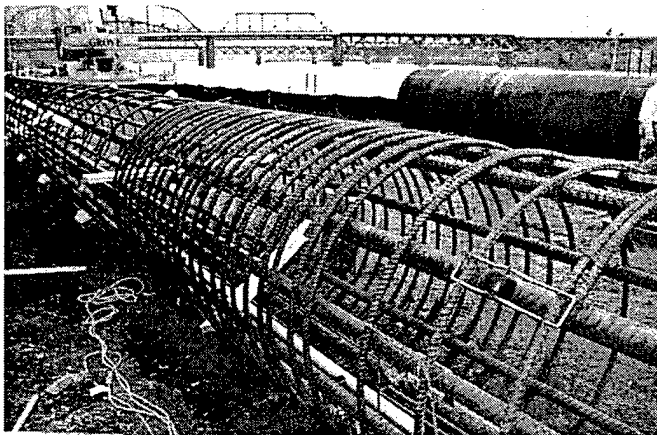


Figure 42. Reinforcing cage for the drilled shaft load test prior to project construction at Braddock Locks and Dam 2

Two additional underwater foundation construction specification sections were added to address specific technical requirements to ensure quality:

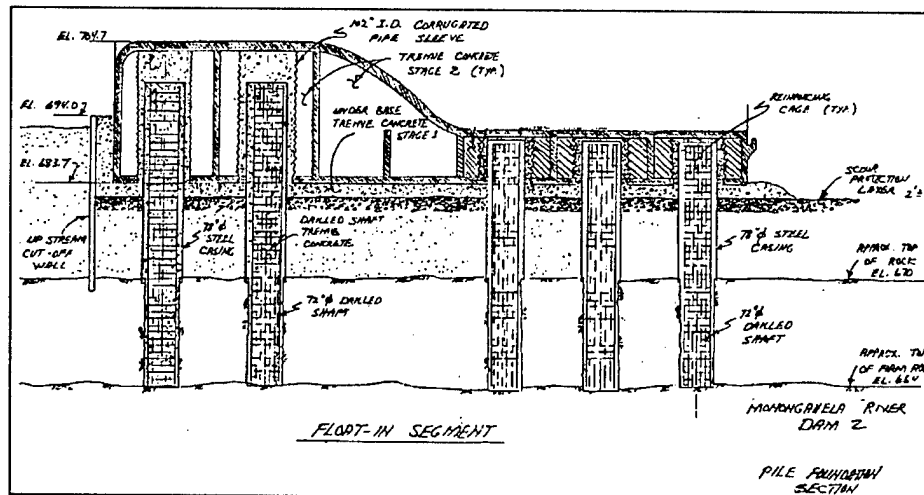


Figure 43. Pinned foundation details, Braddock Locks and Dam 2

Pinned foundation: Exploratory drilling was specified to establish final tip elevations for the bedrock drilled shafts. Exploratory pile-driving served to specify final tip elevations for the alluvium cutoff wall (H-piles, pipe piles, and sheet piles) and to finalize vibratory pile-driving equipment selection for both the cutoff wall piles and the drilled shaft casings. It was specified that these work items were to be completed first, so the contractor would be able to order the correct amount of production piling materials. After the improved-site foundation preparation was completed (i.e., dredging, graded gravel base, and scour protection), a Drilled Shaft Installation Plan was approved. After its approval, a drilled shaft demonstration was conducted to assess and select the equipment and procedures for constructing the production shafts. Previous tests by Pittsburgh District at the site (sheet piles and caissons) provided quality assurance comparisons for both design and construction. Other QC features included the requirement for a fixed-guide pipe pile and casing template and specifying diverless (remote) pipe pile and casing cutoff operations.

Improved-site foundation: Prior to production piling and shaft (caisson) construction, the riverbed was prepared by dredging. Several dredging stages and a graded 12-in. (30-cm) gravel base were specified. Quality control features included surveying specifications and requirements, using a flat-bottom dredging bucket to prevent gouging below final elevation, using a low-velocity suction pump to remove siltation without disturbing the gravel, and diver inspections to ensure siltation removal. Scour protection using specially graded limestone rocks was specified for given areas. Quality control features included laboratory materials testing, diver inspection prior to placement, and disallowing free-fall placement of stones.

New Orleans District. The Inner Harbor Navigation Canal (IHNC) Lock Replacement project will consist of float-in precast concrete elements resting on underwater pinned foundations. Prior to design and construction, a pile load test and installation study contract was awarded. The contract study included installation of nine 48-in. (1.2-m)-diam open-ended steel pipe piles driven to a

maximum depth of 190 ft (58 m) below ground surface. Several innovative features for underwater foundation installation were addressed in this contract.

The technical specifications (U.S. Army Corps of Engineers 1999a) included these sections:

- 02355 Pile Load Test
- 02451 Steel Pipe Piles
- 02497 Dynamic Pile Testing

Contractor QC plans were required to address material delivery, storage, handling, installation, testing, and removal procedures. In addition to material specifications meeting American Society for Testing and Materials requirements, the API (1989) RP-2A-LRFD requirements were specified, and only one splice per pile was allowed. Provisions to monitor horizontal and vertical alignments and plumbness during underwater installation were included, with underwater alignment tolerances of 6 in. (15 cm) vertical, 6 in. (15 cm) horizontal, and within 1.5 percent of length (plumbness).

Only two of the nine pipe piles were specified to be driven underwater in the navigation channel, which is approximately 30 ft (9 m) deep. The remaining seven (onshore) piles were driven inside 6-ft (1.8-m)-diam steel casings in which the soil had been excavated and then backfilled with water to simulate underwater pile driving. All piles and casing were driven with submersible impact hydraulic hammers and/or submersible vibratory (electric or hydraulic) hammers (Figure 44). The piles were instrumented and analyzed using the Pile Driving Analyzer and CAPWAP dynamic load capacity packages.

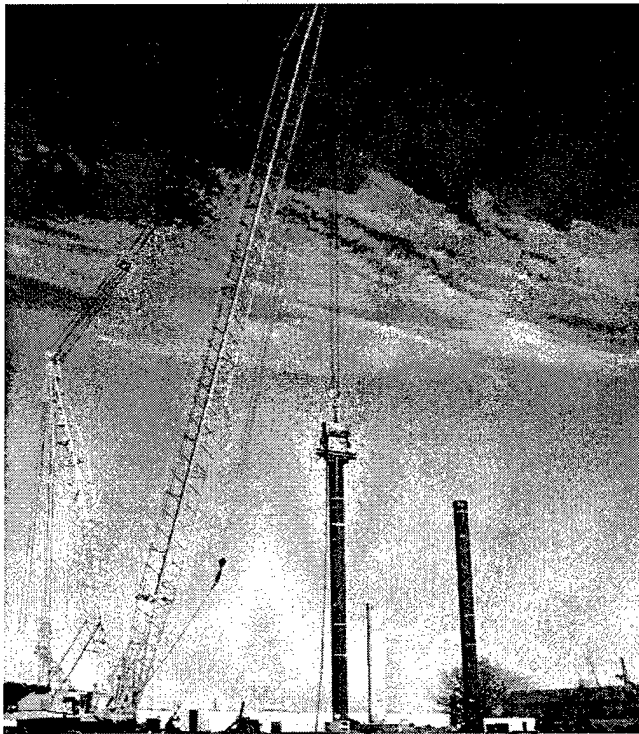


Figure 44. Pile driving with a submersible vibratory hammer, New Orleans IHNC pile test project

Restriking and compression load tests were conducted on five of the onshore piles to determine ultimate pile capacity. Noise and vibration measurements (decibels and inches/second) were specified and taken at two locations during driving of the test piles onshore and underwater for comparison with background readings. Upon completion of the driving and testing, it was specified that the piles and casings were to be removed and the voids filled with a tremied cement-bentonite-sand mixture.

5 Foundation Preparation and Construction

Quality Control and Quality Assurance Issues

Quality assurance and verification on a Corps construction project is a Governmental function that consists primarily of maintaining assurance that the contractor has established and is properly executing the project Quality Control Plan. Verification comes from observing the contractor's adherence to the contract plans and specifications, and includes the contract submittal process, preconstruction meetings, visual inspections, and laboratory testing. The most functional quality assurance program is a reflection of a robust quality control program staffed by competent and experienced contractor personnel.

Numerous training courses are available to help field project personnel fulfill their quality assurance responsibilities. No Governmental training is currently available specifically for underwater foundation construction, but modifications to current training topics may be accomplished. Quality assurance representative's guides such as Engineer Pamphlet 415-1-261 (U.S. Army Corps of Engineers 1992) may also be modified to include underwater construction.

Until the Corps acquires a larger database of underwater construction projects, a successful quality assurance program will depend heavily on the constraints in the quality control clauses of the contractual plans and specifications and the degree to which the specialized contractors execute the quality assurance issues.

Inspection and testing requirements and procedures are highly dependent on the type of underwater foundation being constructed. The contract documents and specifications should cover the inspection and testing requirements during all four phases of underwater foundation construction, as described in the following paragraphs.

Preconstruction phase

After contract award, the preconstruction phase includes specific topics that should be satisfactorily specified, such as preconstruction conferences to establish details of the quality control system, project schedules, and methods for

resolution of potential problems for each phase of the underwater foundation construction. Discussion of the specified and approved Installation Plan is conducted at this time. Topics that should be addressed include potential site-specific problems, such as river (or water body) conditions, concurrent work by others, owner-furnished facilities, owner-furnished horizontal and vertical survey control markers, and coordination, sequencing, and safety issues. Inspection and testing topics should be discussed and understood by all parties to minimize misunderstandings and to reduce potential contract conflicts during construction.

Construction phase

Regardless of the foundation preparation method or the foundation type, a survey positioning system must be established and used throughout the underwater foundation construction phase.

Positional accuracy is required during all phases of the project, from riverbed improvement (including snag removal, dredging, backfill, compaction, leveling, and scour protection activities) to final acceptance. At all locations and during all phases of the construction, three-dimensional (3-D) boundary locations and tolerances must be verified. Floating platforms and supporting items such as drill rigs, pile-driving rigs, and batch plants must be accurately positioned over the foundation site. Mooring with anchor lines or dynamic positioning (propeller-controlled) methods are required to position floating plants and control their locations. The foundation system must be three-dimensionally located and positioned within specified tolerances. Cutoffs, top of surface, and final tip elevations of each foundation element must be accurately measured and recorded. Adequate control measures are required to monitor and control element alignment and plumbness.

Drilled foundations (piles and caissons), especially those that require cast-in-place concrete, require a higher degree of inspection and monitoring because of the high potential for development of unforeseen problems. For drilled foundations, a number of inspection and testing items are important (Greer and Gardner 1986). Items to be considered are listed below and discussed in the following paragraphs.

- Cuttings control and disposal
- Hole bottom cleanout and stabilization
- Subterranean water
- Borehole integrity
- Proof testing
- Reinforcing cage placement
- Concrete placement
- Load testing

Although the objectives are similar, the inspection and testing of underwater drilled foundations require modification of in-the-dry techniques. Once the uncased borehole has been drilled and cuttings have been properly disposed, a cleanout inspection must be conducted to ensure that the bearing stratum is sound

and that the hole is stable. A determination and location of subterranean water such as artesian aquifers must be determined during the site investigation phase as their presence may cause stability problems within the boreholes, which may preclude any uncased cast-in-place concrete.

Remote video monitoring for inspecting the finished borehole is possible if the water is not turbid, or divers may be required to monitor the borehole integrity if allowed in the safety plan. If the borehole is in irregularly weathered rock formations, extra attention should be given to proof testing. Proof testing can be performed by drilling a small-diameter exploratory hole through the bottom of the borehole and noting the advance rate for the purpose of detecting subbottom voids. A probe rod may be jiggled in the exploratory hole to allow detection of sidewall fissures and voids.

The reinforcing cage must be placed with proper alignment and plumbness. Concrete placement requires detailed inspection and testing procedures. Load testing of the completed foundation allows for a comparison of the design and actual strength capacities. Although full-scale load testing is time consuming and expensive, the use of an Osterberg cell may be a less expensive option and would permit the drilled shaft to be placed into service upon completion of the testing (Osterberg 1998).

For driven-pile foundations, Fleming et al. (1992) identified a number of inspection and testing items. A discussion of each is presented in the following paragraphs.

- Quality of pile materials and connectors
- Damage during driving
- Pile heave
- Ground vibration and movement
- Subterranean water
- Workmanship and materials during installation
- Load testing

Although quality control during fabrication is a separate issue, the piling must be inspected for material defects upon arriving at the jobsite. The piling and associated connectors must also be protected from damage from the time they are offloaded onto the jobsite until they have been installed in the ground. Selection of the proper hammer not only involves matching the required load capacity, soil conditions, and piling properties to the hammer, but also includes a requirement to minimize excessive stresses during driving, to prevent damage to the piling by overdriving.

Driving through saturated clays or dense sands may cause pile heave, which is usually of more concern than ground heave, which is caused by soil compaction and consolidation during driving. The potential effects of ground movement on adjacent structures should be considered and addressed during the design phase. Subterranean water such as artesian aquifers may cause problems due to high pore-water pressures if the aquifer is not discovered during the site investigation phase.

All materials, methods, and workmanship on piles requiring additions or modifications during installation, such as grouting, concrete, and welding, must be properly inspected and tested according to applicable codes and standards. Pile load testing during construction may be conducted to permit an assessment of the actual factors of safety (designed capacity/actual capacity). Complete and comprehensive records of the testing setup, procedures, calibrations, adjustments, results, and interpretations are required.

Acceptance phase

Once the foundation system has been constructed, additional inspection and testing should be performed to confirm its acceptance as a finished product. The acceptance phase may be separate or may be conducted during the construction phase, depending on the foundation system. Materials testing conducted during the construction phase may not be finalized until construction is complete (that is, concrete strength tests).

Integrity testing of the completed or installed foundation should be required, and is discussed later in this chapter (see section "Underwater Foundation Testing"). Improved-site integrity testing typically consists of conducting a postconstruction site investigation, which has essentially the same purpose as the preconstruction site investigation.

Gravity-based and pinned foundation integrity testing requires evaluation of the foundation elements (concrete, grout, steel piles, reinforcing, etc.) using standardized material testing methods. The most common integrity testing methods use nondestructive techniques (NDT). For nearly all NDT small-strain integrity testing methods, low-energy ultrasonic pulses or acoustic (sonic) waves are transmitted through the pile materials (concrete, wood, steel, etc.) (Davis and Hertlein 1991).

Postconstruction monitoring phase

After construction has been completed, ongoing inspection and monitoring activities may be continued. These functions may or may not be regulated, depending on the needs of the owner or operator. The Corps requires periodic inspection activities of its civil works structures during their operation and maintenance phase (Department of the Army 1995). Offshore U.S. structure inspections are regulated by the U.S. Geological Survey and the U.S. Coast Guard.

Rules and guidance are also given by the American Petroleum Institute (API) and the American Bureau of Shipping. European agencies such as the U.K. Department of Energy, Det Norske Veritas, and Lloyd's regulate offshore structure inspections outside the United States (Stern and Alia 1980).

The rules and regulations do not place emphasis solely on the structure's foundation. Focus is instead directed to detecting any changes in the overall

structural integrity. To the extent that any loss of structural integrity is due to foundation problems, foundation monitoring may then become a critical component of the inspection process. An important aspect of foundation monitoring is evaluating the foundation material integrity, including steel corrosion, steel weld quality, concrete cracks, concrete spalling, soil scouring, etc. It may also be important to evaluate the foundation performance in terms of settlement, inclination, stability, etc. Unique foundations such as suction caissons, which depend on active pore pressure monitoring, may require a special instrumentation and evaluation process (Tjelta 1993; Stock, Jardine, and McIntosh 1993).

Various tools and equipment are available for conducting foundation inspections. Remote visual monitoring may be accomplished with remotely operated vehicle systems (ROVs) or autonomous underwater vehicles similar to those used for underwater site investigations. Diver-equipped tools are also available for material integrity evaluations, including high-pressure waterjets for cleaning steel and concrete and hand-held equipment for inspecting concrete and steel components (Smith 1987).

Foundation Installation and Testing

A variety of installation procedures and equipment exists for installation of underwater foundations. Testing procedures and equipment are required to validate the foundation design and to ensure foundation integrity.

Improved-site foundations

Improved-site foundations include those that are modified to increase strength, reduce settlement, and improve stability of the soil or rock. Physical, mechanical, hydraulic, and chemical methods are available to achieve these modification objectives (Hausmann 1990). Physical modification includes excavation, backfilling, and leveling operations. Mechanical methods generally include vibratory compaction, confinement, inclusions, and geotextile reinforcement. Hydraulic methods include the use of vertical and horizontal drainage systems. Chemical modification involves deep cement mixing.

Physical modification.

Excavation and dredging. Both terms relate to the removal of undesirable soil or rock materials and are often used interchangeably. Excavation generally implies a smaller operation, such as removing soil inside a cofferdam. Dredging implies a larger area operation and is usually required during installation of improved-site foundations and gravity-based foundations. Reasons for excavation and dredging include removing unsuitable materials such as loose/weathered rock or mud overburden, cutting specified depths and slopes in competent materials, retrieving bulk samples, and cleaning (removing) soft bottom

(sediment) materials after dredging or backfilling operations have been completed (Johnson, Compton, and Ling 1972).

Dredging to remove overlying soft materials may be accomplished using airlifts, suction (hydraulic) dredges, or specialized trenching equipment. Dredging firmer materials may be accomplished using clamshell (orange-peel) draglines (Figure 45) or bucket (dipper) dredges in shallower water. Deepwater dredging is done with continuous dragline, cable with attached cutters, ocean dredging vessels (suction hydraulic or hopper with cutterheads), or ROVs. Explosives are used to remove pinnacles and boulders, and drilling/blasting operations are performed in rocky areas (Gerwick 1974). Harris (1942), Tate (1961), Millard and Hassani (1971), Johnson, Compton, and Ling (1972), and Loeken (1980) describe dredging operations on numerous projects, including the Navy's drydock projects, New York's Yonkers sewage treatment plant, San Francisco's marine terminal, Hong Kong's Plover Cove Dam, Ghana's Akosombo Dam, and offshore borrow areas.

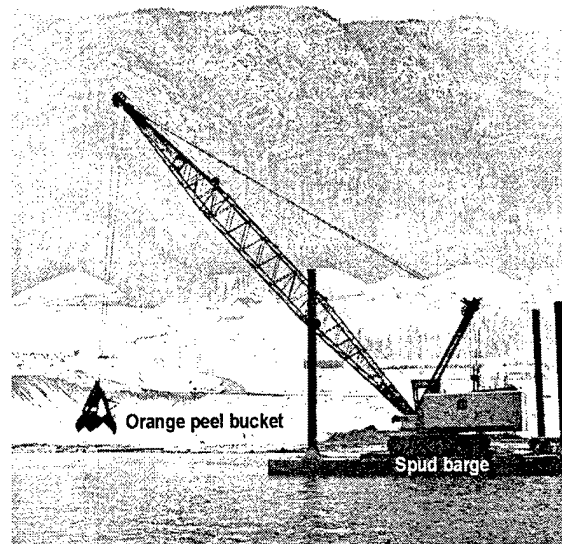


Figure 45. Clamshell excavation and soil sampling in the Columbia River (after Bazett and Foxall 1972)

Large-scale dredging and backfilling operations may use oceangoing vessels. Large ocean going trailing suction hopper dredges have been specially designed for removing and backfilling bottom materials at depths to 430 ft (130 m). These dredges pump slurried bottom material into vessel hoppers via suction pipes connecting the bottom-trailing dragheads to the vessel's main dredge pumps. The dredge then transports the material to the disposal location, where the hopper load is discharged for placement of underwater fill or is wasted. Currently, the world's largest trailing suction hopper dredger is Belgian Jan De Nul's with 39,200-cu ft (30,000-cu m) hopper capacity (Dredging and Port Construction 1999). Smaller dredges are available for dredging near-offshore and inland rivers to depths necessary for maintaining navigation, usually less than 50 ft (15 m). Submerged dredging platforms allow access to deeper locations, and are typically used offshore. One Netherlands company has developed an ROV capable of dredging to depths of 3,300 ft (1,000 m), as illustrated in Figure 46 (Van Oord 1999). Specialized subsea ROV trenchers for pipeline burial are also available (Rushfeldt 1981, Dansette and Robertson 1994).

Positional accuracy for dredging operations has been improving thanks to modern sensor technology and global positioning system (GPS). One example is Prolec's "Digmaster" indication system from the United Kingdom. With its sensors mounted on a bucket (dipper) dredge, the operator can visually monitor in real time where the bucket is, and dredging tolerances to 2 in. (5 cm) are

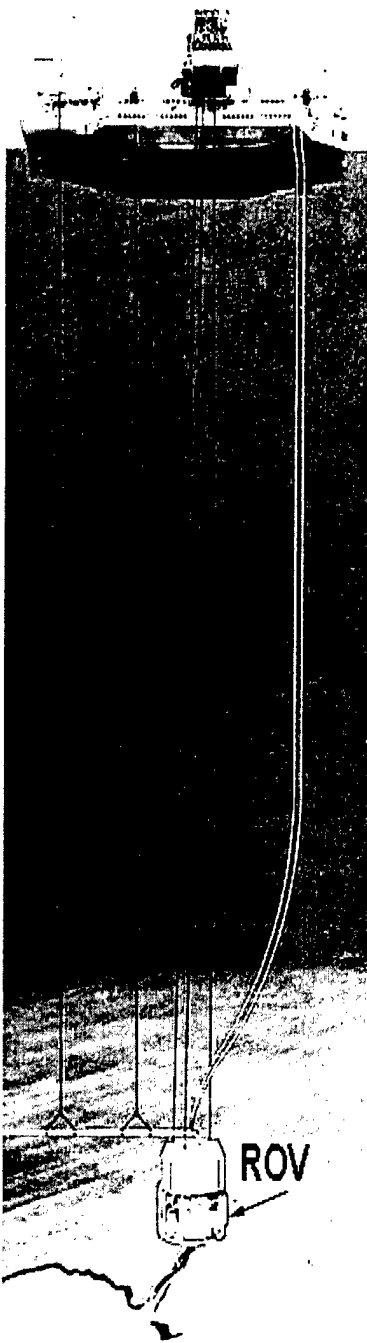


Figure 46. Deep-sea dredging operation

possible when dredging at depths to 60 ft (18 m). Tidal and current fluctuations are automatically taken into account. Prior to the development of this technology, the dredging operator relied on boom markers and hand soundings (*World Dredging, Mining, and Construction* 1999).

Small quantities of material are removed by mechanical ejectors operated locally by divers or remotely from a work barge. Mechanical ejectors (also called “air lifts”) operate using compressed air and water (Figure 47). They work by loosening the material at the base of the excavated area, mixing it with the lighter air bubbles, and discharging it to the surface.

Ejectors do not work well in soft clays or clayey silts as they merely form a hole at the base of the excavated area and extra work is required to slump these materials toward the ejector (Tomlinson 1994). Air lift methods were used on the Thames River flood barrier caissons to remove sediments and excavate chalk that adhered to the cofferdam sheet piles and walers.

Explosives were placed by divers to dislodge the chalk deposits prior to excavation (Gilbert and Horner 1984). A 10-in. (25-cm)-diam airlift operated with a 1,000-cfm (30-cu m/min) air compressor successfully cleared boulder intrusions in rock sockets 100 ft (30 m) underwater at Bantry Bay (Fox 1970).

Underwater air bubblers were used during underwater backfill operations to prevent “density currents” of suspended fines that flowed upstream and downstream on the river bottom during construction of the submerged foundation for a dam in British Columbia (Bazett and Foxall 1972).

Fill placement. After the unsuitable material has been removed, the site may be further improved by backfilling, leveling, and in situ densification (e.g., soil strengthening).

There are numerous case histories and examples of underwater improved-site foundations using various equipment and methods that can be obtained for additional details.

Underwater fill placement (also called backfill placement if previously dredged or excavated) can be accomplished using several methods (Johnson, Compton, and Ling 1972; Bazett and Foxall 1972; Loeken 1980; Mitchell and McRae 1985; DOSBOUW 1987), as described below.

Methods for underwater fill placement include:

- a. Land-edge dumping by bulldozer, backhoe, or clamshell bucket
- b. Placement from deck scow (barge) with buckets or dozer
- c. Hydraulic slurry mix pumping or jetting
- d. Dumping by bottom-dump scow or vessel
- e. Fall-pipe placement

When selecting the optimal method, numerous variables must be considered, including project size, water depth, material type (gradation), final slope requirements, current velocity, environmental considerations, borrow material location, and economic costs.

Figure 48 shows an example of the backfill material gradation curves selected for an underwater pipeline project.

Bottom-dumping scows require a minimum depth of about 15 ft (5 m) because of scow and tug drafts and propeller disturbance. Material placement is relatively fast, and flat slopes are usually obtained (unless retained). At the Southern Pacific Railroad crossing of the Great Salt Lake, bottom-scow placement of a well-graded silty sand achieved slopes inversely proportional to the water depth. Maximum slopes (1:2) were achieved at shallower depths but tapered off to 1:10 slopes at 40-ft (12-m) depths (Johnson, Compton, and Ling 1972). Bottom-scow placement of glacial till material at the Hugh Keenleyside Dam project caused segregation and loss of fines, requiring design changes. Initially, dumping was restricted to river currents

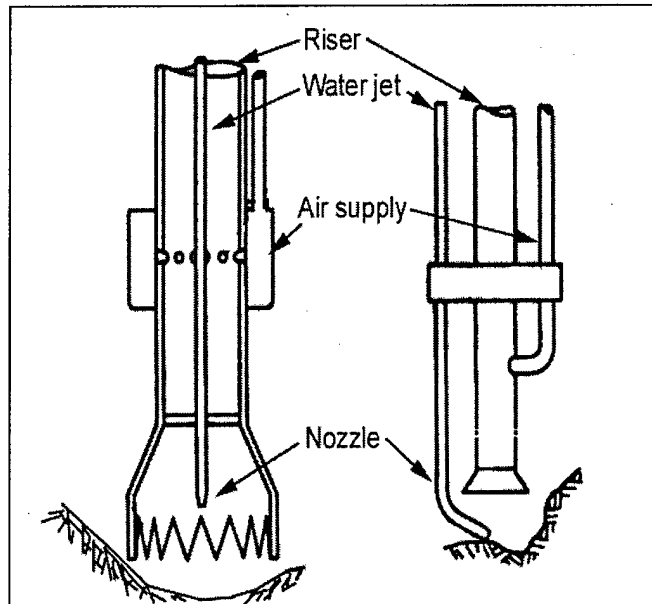


Figure 47. Two types of underwater ejectors (after Tomlinson 1994)

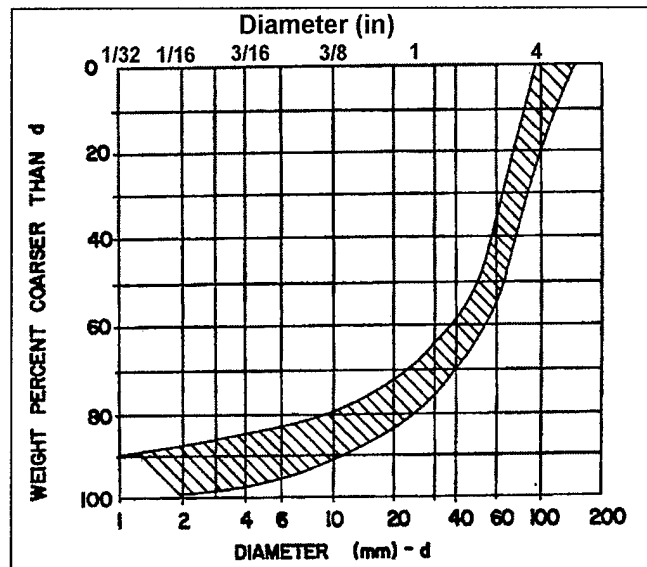


Figure 48. Graduation curve for underwater pipeline stable backfill material

less than 2 ft/sec (0.6 m/sec) to avoid loss of fines. However, it was determined that suspended fines flowed upstream along the riverbed bottom, and turbidity increased at this low velocity. To help alleviate the problem, dumping in river currents up to 4 ft/sec (1.2 m/sec) was allowed, and underwater dykes were installed (Bazett and Foxall 1972). Materials containing entrapped air and dumped suddenly from bottom-dump scows appear to liquefy as the material hits the bottom. For example, Johnson, Compton, and Ling (1972) observed a rapid bottom-dumped sand spreading horizontally at a rate of 20 ft/sec (6 m/sec).

Deck scows are usable in shallower water and enable steeper backfilled side slopes. Tate (1961) reported that slopes of 1:2 were obtained by hydraulic jetting of well-graded material (3 in. down to a maximum of 10 percent finer than the No. 100 sieve) from a deck scow. Johnson, Compton, and Ling (1972) reported the placement of rock by clamshell bucket off a deck scow at the Hong Kong Plover Dam site. The bucket was opened just above the bottom to minimize segregation. Pore-pressure measurements were monitored and indicated a period of 2 months was required for dissipation of pore pressures in the fill materials.

Underwater hydraulic fill placement may cause coarser materials to drop out first, which could lead to unstable slopes. Experiments by Mitchell and McRae (1985) determined that slurry-pipeline placement of sand obtained lower densities than those achieved with bottom-dumping hopper dredge systems. Placing the sand in "layers" and compacting with a special air gun achieved relative density increases up to 80 percent. Without underwater compaction efforts, the relative densities of the sand ranged erratically from about 10 to 70 percent (Johnson, Compton, and Ling 1972). Stewart, Jefferies, and Goldby (1983) reported that the average relative density (without compaction) for a uniform sand (D_{50} approximately 0.28 mm and fines less than 5 percent) placed underwater was about 40 percent.

Underfilling of structures such as immersed tube tunnels and offshore gravity-based structures has been accomplished using hydraulic fill variations. Typically, a mixture of sand and water is pumped between the structure bottom and its supports to (a) provide a good structure/seabed contact and (b) to create a uniform distribution of contact stresses. For offshore structures, a grout mixture is typically used instead of a sand slurry, although a sand slurry underfill enables easier remobilization of the structure. Sand slurry underfill has been used frequently in the Beaufort Sea mobile oil drilling platforms (Buslov et al. 1984).

The fall-pipe placement method generally allows precision placement of coarse materials to deeper depths with minimum segregation. Offshore precision fill placement equipment currently consists of the fall-pipe method used by offshore dredging companies such as Tideway BV, HAM, Ballast Needam, and Boskalis. One earlier project (during the 1970s) that used two types of fall-pipe systems was a North Sea pipeline backfilling operation. A fine sand layer was covered by a gravel layer (4-in. or 10-cm maximum particle size). The materials were placed in water with bottom currents up to 8 ft/sec (2.5 m/sec) at depths up to 230 ft (70 m). One fall-pipe system had remotely controlled electrohydraulic thruster units attached for accurate placement of the conveyed gravel. The other system pumped hydraulic fill (sand and gravel) through the fall-pipe. The

positioning of the fall-pipe was accomplished using an underwater acoustic referencing system (Loeken 1980). Stewart, Jefferies, and Goldby (1983) reported another project in which a modified drag arm on a trailing suction hopper dredge was used to discharge a sand slurry 8 ft (2.5 m) past the arm end to the deposition point. Average slopes of 1:5 were placed in 100-ft (30-m) water depths. Smith (1962) reported that a 2-ft (0.6-m)-diam steel fall-pipe was used to place gravel (8-in. or 20-cm maximum size) at the base of a Mississippi River dam.

Leveling. If the superstructure is founded on an improved-site or gravity-based foundation, the bottom must be leveled to specified tolerances after dredging and/or backfilling. Typical out-of-level tolerances were described by Gerwick (1974) as being about 2 in. (5 cm) on hard bottoms to 12 in. (30 cm) on softer bottoms (sand and silt). Gerwick also stated that most leveling sequences include the following steps:

- a. Placement of a crushed rock base (similar to the fill placement noted above). A well-graded crushed (not natural gravel) rock base course can be placed by barge bottom-dumping (effective only to about 100-ft (30-m) depths), through a fall-pipe, or lowered in buckets. Natural gravel is not desirable due to its tendency to displace laterally (slide) under load. Typical rock base courses use a maximum aggregate size of about 6 in. (15 cm).
- b. Placement of a "screed course" on top of the rock base. If desired, a smaller layer up to 40-in. (1-m) thickness with maximum aggregate size of 3 in. (7.5 cm) can be placed on top of the rock base to enable more accurate screeding (of the smaller particle sizes). Typically, the screed course is placed using a tremie pipe (fall-pipe).
- c. Level (screed) the aggregate. Many types of screeding devices have been used on various underwater projects. All are designed to provide stable methods for leveling within the specified tolerances. The historical method employed for shallow-water depths used a heavy steel beam that was dragged across the bottom from a surface platform. Other devices are manually operated underwater and require divers. Self-leveling frames with remotely controlled positioning devices are used in deeper water.

Dredging equipment has also been used to provide leveling and trimming of the base course. Boskalis (1999) developed a multipurpose pontoon system ("Scradeway"), which allowed multimillimeter accuracy for placing, leveling, and trimming the gravel bed prior to sinking the Oresund's immersed tube tunnel elements.

Mechanical modification. Mechanical modification by vibrocompaction increases the relative density of granular deposits. To accomplish this increase in density, the vibrating energy is used to rearrange (and density) the soil structure. The vibrating energy is transferred to the soil via a shaft attached to a floating platform system. Typically, inside the shaft is a hydraulic motor driving an

eccentric (off-centered) mechanism that vibrates due to its eccentricity. The remainder of the shaft is isolated to prevent vibrations beyond the shaft tip. Water may be jetted through the shaft tip to enhance soil consolidation and shaft penetration (vibroflotation). Vibroflotation has been successfully used for increasing the bearing capacity of previously submerged cohesionless materials.

Examples include the underwater placement (but subsequently dewatered) fills such as Wanapum Dam (Engstrom 1963) and Bremerton drydocks (Tate 1961). Vibratory compaction using pipe piles was performed on dewatered fill near Baltimore (Millard and Hassani 1971). Figure 49 shows a vibratory compaction probe. Figure 50 shows a specialty pontoon containing four similar vibratory compaction probes.

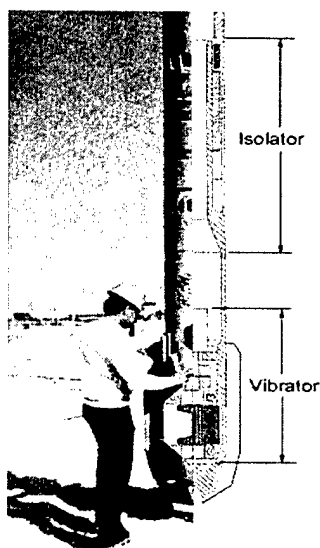


Figure 49. Vibratory compactor (after Bauer, Inc. 1994)

Several worldwide contractors use specialized equipment for mechanical modification. Vibrocompaction equipment may be specified simply to achieve vibratory compaction, or it may be combined with other proprietary systems to construct pinned foundations without soil excavation.

Underwater stone columns have been placed for the purpose of increasing the soil's shear strength and bulk modulus. Pennine, Ltd., (1998) reported the installation of 2.6-ft (0.8-m)-diam by 66-ft (20-m)-long stone columns in water depths up to 108 ft (33 m) from a floating barge using vibrocompaction, as illustrated in

Figure 51. The productivity rate was greater than one column per hour, including barge positioning.

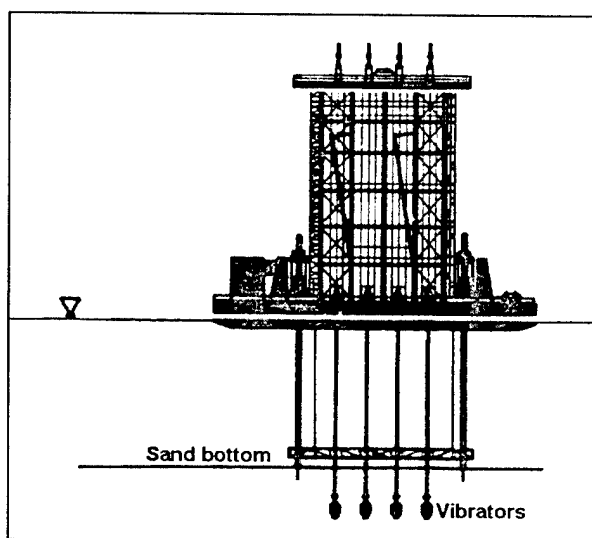


Figure 50. Vibratory compaction pontoon *Mytilus* used in the Eastern Scheldt project (after DOSBOUW 1987)

Hayward Baker (1999b) reported the installation of 270 stone columns (3-ft (1-m)-diam) using a vibroreplacement method through water depths up to 40 ft (12 m) to prevent liquefaction damage to existing underwater piles at a Vancouver port facility.

According to Stewart, Jefferies, and Goldby (1983), underwater self-consolidation of cohesive material (natural or backfilled) is not typically used because of (a) monitoring instrumentation limitations, (b) insufficient experience, especially with hydraulic fill/clay separation, and (c) limited large-scale experience relying on consolidation for strength gain.

Another mechanical modification technique consists of overlying the soil with prefabricated blankets, mats, or geotextile reinforcement. Typically, the overlays are constructed of steel, concrete, graded soil materials (sand, fine gravel, and coarse gravel), or synthetic fabrics. The purpose of the blankets is to reduce the induced bearing pressure on the subsoil and to prevent erosion of the subsoil beneath the mat. Prefabricated mats were used extensively in the Eastern Scheldt project (DOSBOUW 1987). Specialized mat-sinking equipment barges (Figure 52) were required during this operation. Immediately prior to placing the mats, the sand bottom was redredged to remove any silt sedimentation. Bottom compaction was then accomplished with four ICE-815 underwater vibratory hammers each mounted on a perforated 13- by 13-ft (4- by 4-m) steel plate and dragged in tandem fashion across the dredged surface (Jonker 1987). The prefabricated mats were then placed over the vibrocompacted sand.

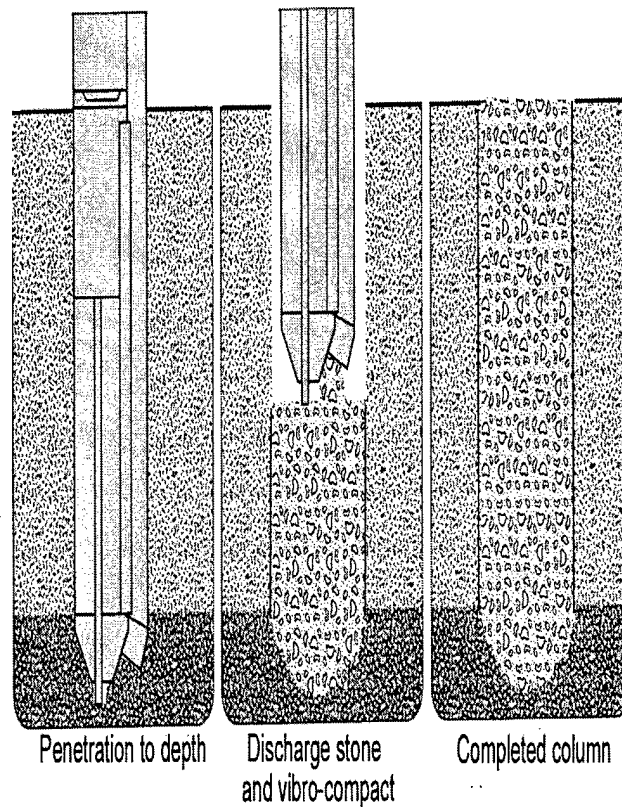


Figure 51. Vibratory stone column installation (after Pennine, Ltd., 1998)

Abbott and George (1989) selected steel reinforcement meshes for their improved-site foundation. Flat galvanized steel strips were placed 9 in. (23 cm) on center and covered with granular backfill. Specialized methods were required for the underwater placement of the reinforcement. Onshore reinforced earth applications allowed the reinforcement mesh steel strips to be singularly placed, but underwater installation required a mat arrangement that was constructed onshore and then lowered into the water as a complete mesh mat. Granular backfill (angular coarse to medium sand and gravel) was placed on the mat arrangement.

Steel mudmats have been used offshore to increase the bearing capacity of soft bottom materials and are frequently attached to the base of jacket structures. The thin (typically less than 0.5-in. or 1.5-cm) mudmats provide stability and tip-over resistance after jacket setdown and prior to pinned foundation installation. Improvements to resist lateral sliding include perimeter skirts and interior ribs. Digre et al. (1989) discussed design details for the Bullwinkle platform skirted mudmats. Lieng and Bjorgen (1995) discussed a release system for mudmats on a Heidrun platform subsea structure.

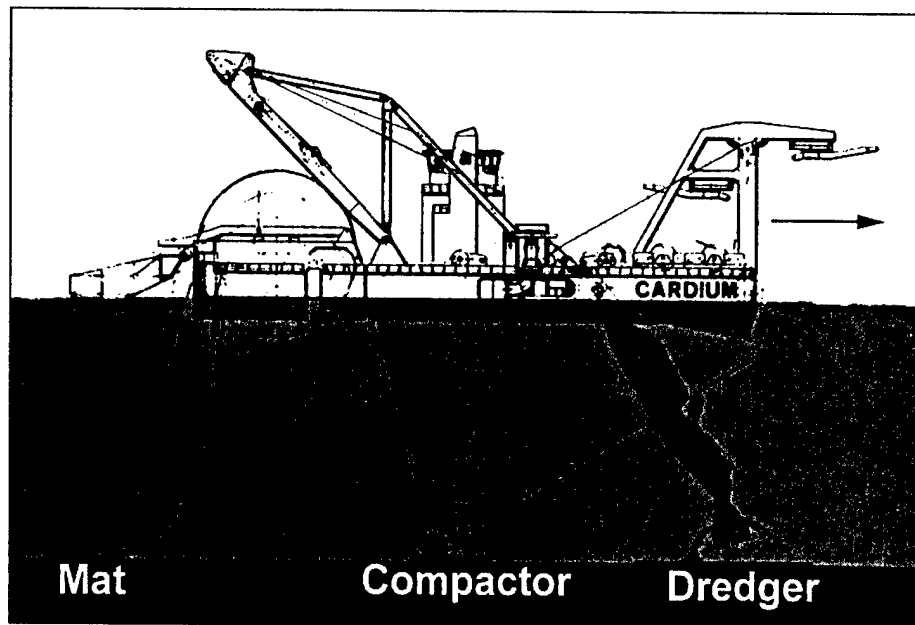


Figure 52. Eastern Scheldt mat-sinking pontoon (after DOSBOUW 1987)

Hydraulic modification. Hydraulic methods use vertical or horizontal drainage systems to remove excess pore fluid from the soil matrix. Vertical drainage systems (wick drains) require a surcharge or overpressure method for generating excess pore-water pressure. Several proprietary wick drain systems are available, including those from the Bauer, Geotechnics America, and American Wick Drain Corporations. Horizontal drains require a vacuum (underpressure) source to remove excess pore-water pressure. For horizontal drains, it is critical that leakage from the overlying water body not be permitted. Water drained as a result of the underpressure method is pumped away from the site (Aerts, Devlieger, and Vandycke 1999).

Chemical modification. Offshore chemical soil modification with deep cement mixing has been successfully used in Japan and the Arctic oil region for several years. It was developed in the United States in 1951 by the U.S. Navy, but was discontinued in the United States for economic reasons (Halebsky and Wetmore 1986). It has been demonstrated to improve soft soil unconfined compressive strengths by up to 700 percent.

Deep cement mixing barge-mounted systems (similar to a system illustrated in Figure 53) have been operated in water depths to 230 ft (70 m) for mixing soil 165 ft (50 m) below the mudline. The process consists of pumping cement slurry under pressure into the seabed followed by in situ soil mixing. Each mixer can improve a soil area up to 62 sq ft (6 sq m) per penetration. Normal slurry mix is a 0.6 water-cement ratio. Typical cement concentration is about 10 to 20 percent, which is approximately 10 to 20 lb of cement per 100 lb of dry soil (Halebsky and Wetmore 1986).

Underwater grouting using a barge-mounted drill was described by Hayward Baker (1999b). A 3-in. (7-cm)-diam grout pipe was advanced below a sanitary sewer tunnel under a pond. Compaction grouting strengthened the soft organic soil, allowing tunneling to be completed using a tunnel boring machine.

Quality control. Quality control and quality assurance (QC/QC) testing is conducted to determine the adherence to design requirements, as stated in the contract specifications. For the improved-site foundation, the testing basically consists of comparing the before, during, and after construction conditions to determine if the specified improvements were achieved.

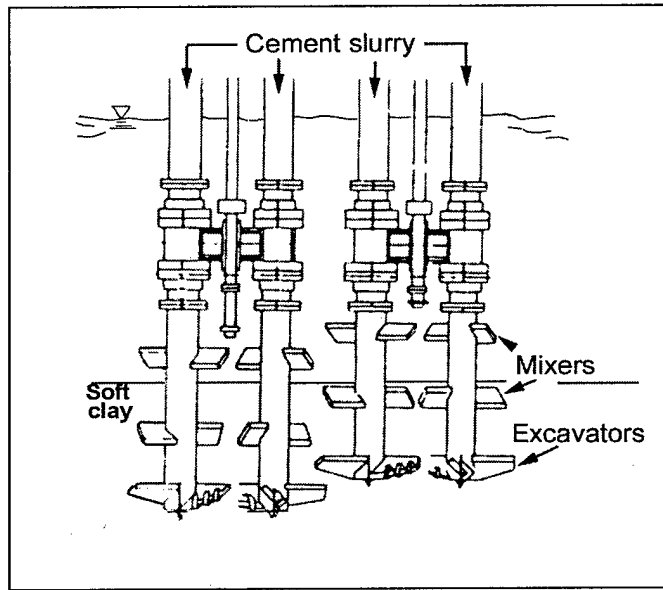


Figure 53. Deep cement mixing system (after Halebsky and Wetmore 1986)

Quality control during dredging for improved-site foundations includes (a) material inspections, (b) surveying for depth, distance, and volume measurements, and (c) postdredging cleanup of any remaining soft sediments (Johnson, Compton, and Ling 1972). Material inspections include bottom examinations by divers or remote monitoring, visual observations of the dredged materials, sampling of the bottom materials and/or the dredged materials, and testing. Site positional control is accomplished by hydrographic surveying methods and is a prerequisite for foundation construction. Ensuring that dimensional tolerances for volumetric quantities, areal extent, and cut slopes are not exceeded is one function of the QC/QA process. It is economically important that accurate surveys are conducted to establish the lateral dredging boundaries and that material inspections are conducted to define the depth of cut, when unsuitable materials are being removed (Johnson, Compton, and Ling 1972). Stewart, Jefferies, and Goldby (1983) discuss quality control methods for dredging operations conducted to remove unsuitable material, including core sampling of the dredged material inside the hoppers.

Postdredging cleanup of remaining soft sediments is a critical element that should always be expected and performed. Inspection by divers or sampling should be conducted to determine the presence and extent of remaining or redeposited sediments (Johnson, Compton, and Ling 1972).

Quality control considerations during fill placement and leveling operations are similar to those for dredging operations. Material inspections and surveying are required to ensure that the proper materials are being deposited at the correct location and are being placed according to specified thickness and side slopes.

The unforeseen presence of fines (namely, silts) in cohesionless fill materials causes quality control problems and was discussed by Johnson, Compton, and Ling (1972), Bazett and Foxall (1972), Stewart, Jefferies, and Goldby (1983), Buslov et al. (1984), Mitchell and McRae (1985), and Jonker (1987). The loss of fines in cohesive fill materials and corrective measures to control it was discussed by Bazett and Foxall (1972).

Integrity and/or performance tests are necessary to ensure that specified engineering properties such as shear strength and relative density have been met. These should be conducted during and after the fill placement or site modifications, and may include in situ tests and sampling operations. In situ tests include SPT, CPT, PMT, vane shear, and geophysical techniques. Sampling can be accomplished by bucket, probe, cores, or other equipment. Bazett and Foxall (1972), Stewart, Jefferies, and Goldby (1983), Mitchell and McRae (1985), and Jonkers (1987) discussed density measurements and control. Bazett and Foxall (1972), Johnson, Compton, and Ling (1972), and Buslov et al. (1984) discussed aspects of sampling and testing cohesive materials. Figure 54 illustrates the use of the CPT as a measure of in situ density for quality control during construction of the Eastern Scheldt project.

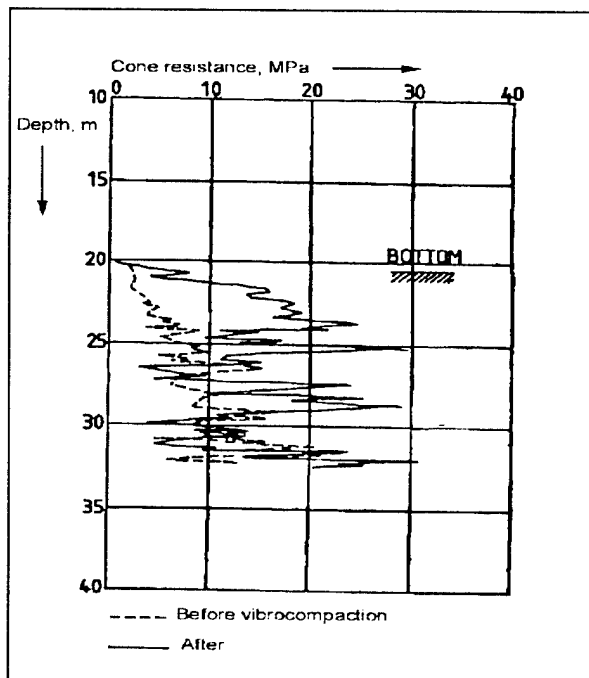


Figure 54. Eastern Scheldt, CPT results—before and after vibrocompaction (after Jonker 1987)

Gravity-based foundations

Since most gravity-based foundations are floated or lifted in, the installation procedures include towing and positioning concerns (heavy-lifting equipment availability, weather, positioning accuracy, etc.). Once the foundation or structure is positioned for placement, the following concerns become critical (Hove and Foss 1974):

- a. Wind, wave, and current limitations.
- b. Installation sequence.
- c. Touch-down load concentrations.
- d. Leveling and/or base grouting.
- e. Installation mating procedure with superstructure elements.

Numerous case histories are cited in the offshore industry literature which detail installation considerations for float-in and lift-in gravity-based foundations and structures (Gerwick 1974; Hove and Foss 1974; Alloni, D'Agostino, and Priarone 1976; Collipp and Johnson 1979; Noblanc and Schnader 1983;

Berthin et al. 1985; Tateishi and Watanabe 1986; Tanaka et al. 1987; Broughton and den Hertog 1990; Christophersen 1993; Alm, Bye, and Egeland 1995; Andenaes, Skomedal, and Lindseth 1996; Waddell 1997).

Gravity-based foundations that are cast in situ (using tremied concrete or grout) also require special installation considerations for the formwork, rebar, and concrete placement. References and case histories for formwork placement and tremie concrete techniques include those from Harris (1942), Tate (1961), Hedefine and Silano (1968), Johnson, Compton, and Ling (1972), Gerwick (1974), Becker et al. (1998), and Yao, Berner, and Gerwick (1999).

Pinned foundations

Driven piles. Pile foundations are installed using impact-driven, vibration-driven, jacking, jetting, suction, or drilling techniques, or combinations of these. The most common method of installing onshore piles is dynamic impact from a variety of hammer types. Pile-driving hammers include the simple drop hammer, single-acting hammer, double-acting hammer, differential hammer, and the diesel hammer (ASCE 1984). Hammer energy is supplied by air, steam, hydraulic pressure, diesel, or electricity. Vibratory hammers produce dynamic forces by internal counter-rotating masses using air, steam, electric, or hydraulic energy sources. Jetting, jacking, and screwing methods are specialized, and their usage in underwater piles has not been widely found in the literature. The majority of underwater piles have been installed using impact-driven methods, although vibratory-driven methods are gaining in popularity for offshore operations. The principal reason that offshore vibratory hammers are not as popular as impact hammers is due to the larger offshore pile weights, which require greater vibration energy (Jonker 1987).

Impact hammers. Offshore piles driven with above-water steam-impact hammers dominated through the 1970s. Pipe piles ranging from 30 to 48 in. (75 to 120 cm) in diameter with nonsliced lengths from 200 to 350 ft (60 to 110 m) were routinely driven (total spliced pile lengths) up to 800 ft (240 m) below the mudline in the Gulf of Mexico. Offshore pile hammers (typically single-acting steam-driven) with rated energies from 60,000 to 300,000 ft-lb (80 to 400 kNm) were used to drive these piles to their design penetrations. The resulting ultimate pile load capacities were up to 3,500 tons in compression and 2,000 tons tension (Sullivan and Ehlers 1972). Pile-driving experience in the hard clays of the North Sea caused overestimation of pile hammer capabilities and underestimation of the pile bearing capacities (Fox, Parker, and Sutton 1970). Offshore (above-water) pile hammer manufacturers included Vulcan (Figure 55), MKT, Delmag, and Menck. Steam hammer energy capacities increased to 600,000 ft-lb (810 kNm) by the mid-1970s.

Development of the submersible hammer in the mid-1970s opened up new frontiers in pile-driving history. Pile followers, steam lines, and costly anchored derrick ships were no longer needed. The Hydroblok hydraulic hammer (by HBM, Hollandsche Beton Groep N.V.) operated an internal piston using nitrogen gas supplied through an umbilical cord to the underwater cage attached to the

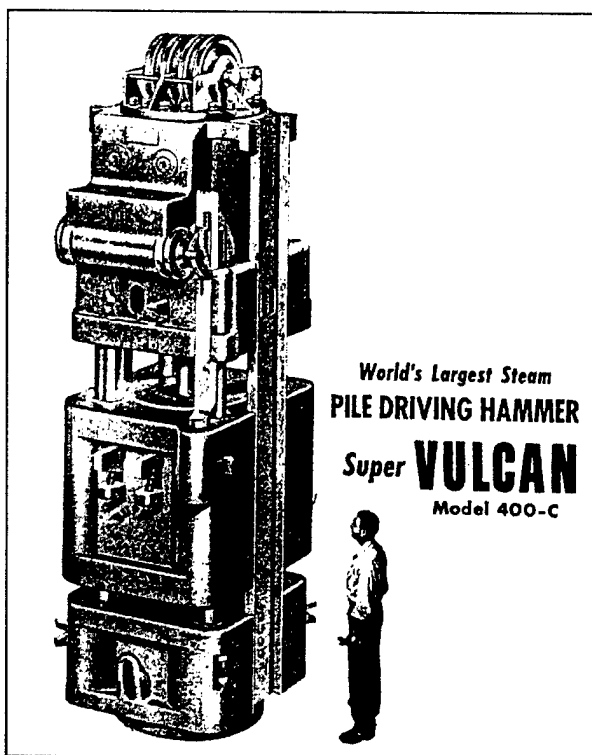


Figure 55. Vulcan steam hammer advertisement from 1957 *Civil Engineering* magazine

pile top. The rated energy of the HBM 3,000 was 800,000 ft-lb (1,080 kNm). It could be used to drive an 84-in. (2.1-m)-diam pipe pile (Jansz et al. 1976). Cox and Christy (1976) and Jansz (1977) reported that this underwater hydraulic hammer successfully drove piles in the Gulf of Mexico and the North Sea. In hard clay locations, a method to vertically align piles topped with the HBM hammer was devised.

Alignment and lateral support problems of submerged pile hammers were addressed (Jansz and Brockhoff 1979). For the Eastern Scheldt project, an HBM 1500 hammer was used for driving mooring anchor piles. Sitter (1980) reported the development of an underwater hammer (HBM 4000) with rated capacity of over 1 million ft-lb (1,350 kN-m) of energy. The Menck company, which previously had made steam hammers, began making underwater hammers (such as the

model shown in Figure 56). Heerema (1980) reported that a patent conflict developed in the late 1970s between HBM and Menck.

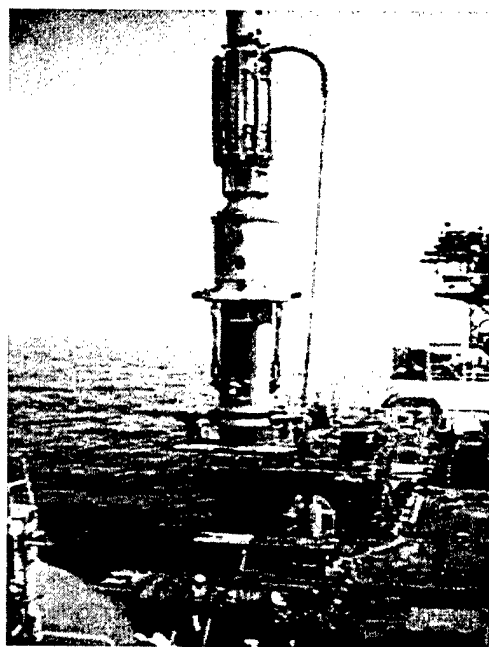


Figure 56. Menck 500T underwater hammer (after Menck 1999)

Another submersible hammer was manufactured by the Raymond Company (RU-300). Its rated energy was 300,000 ft-lb (404 kNm). Its first offshore assignment was off the coast of New Zealand on the Maui jacket platforms that have 48-in. (1.2-m)-diam open-ended pipe piles driven 230 ft (70 m) below the mudline (Gendron, Holland, and Ranft 1978; Rennie and Fried 1979).

Several other innovative pile-driving methods were introduced during the late 1970s. These developments include the submerged water hammer pile driver, which used seawater pressure to create a cyclic water-driven impact mechanism to drive the pile (Wisotsky 1978), and electro-osmosis, which used direct current and achieved considerable pile-driving resistance reduction in clay

deposits due to the soil chemistry bond force reduction (Rose and Grubbs 1979).

Slender hydraulic underwater hammers (Figure 57) were introduced in 1980 by Menck, Inc. By 1983, the hammers were instrumented with an adequate internal monitoring system comparable to the above-water hammers (Van Zandwijk 1986). Slender hammers proved to be much more versatile for offshore jacket pinning than were the larger diameter first generation of underwater hammers. Currently, underwater hammers manufactured by Menck GmbH (Germany) and IHC Hydrohammer BV (Netherlands) are used for offshore applications.

Both manufacturers build hydraulic hammers for underwater applications. Menck's largest hydraulic hammer (MHU 3000T) has a rated energy of 2.4 million ft-lb (3,300 kNm) and weighs 400 tons. IHC's largest hydraulic hammer (the S-2300, shown in Figure 58) has a rated energy of 1.7 million ft-lb (2,300 kNm) and weighs 260 tons. Piles up to 102 in. (2.5 m) outside diameter may be driven with these hammers (Menck 1999, IHC 1999).

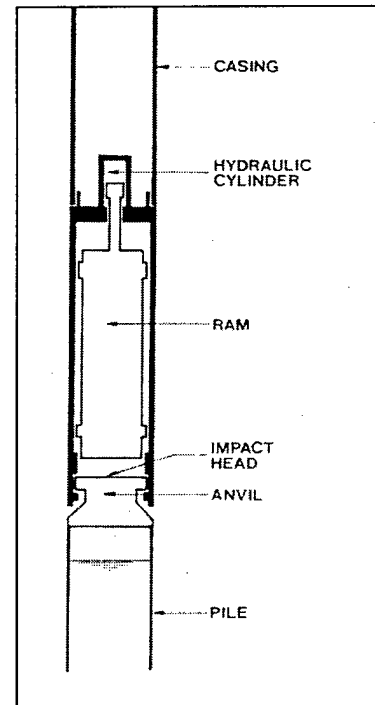


Figure 57. Underwater hydraulic slender hammer (after Van Zandwijk 1986)

Soil resistance to impact-driven piles. Driving piles to their designed penetration lengths may not always be possible due to the inadequacy of the pile-driving equipment in overcoming soil resistance. The analyses of

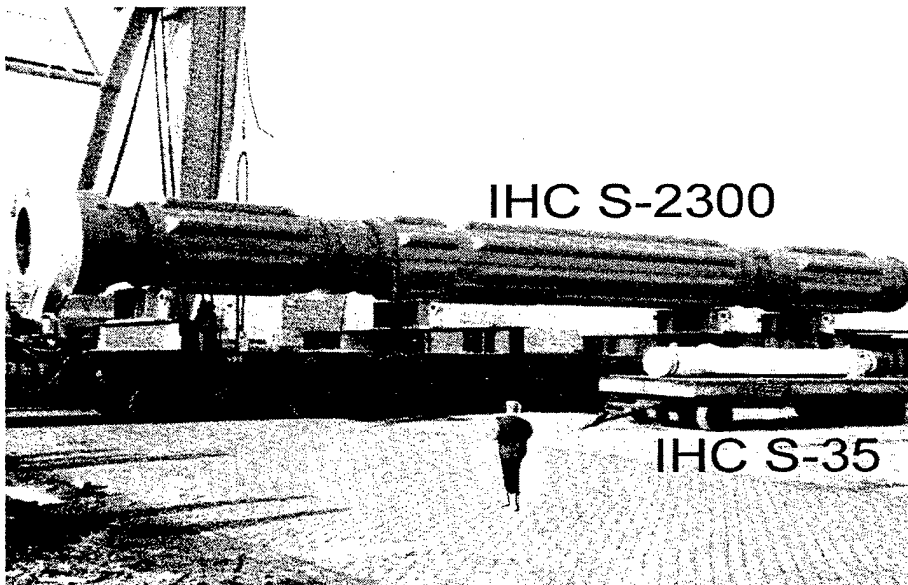


Figure 58. IHC underwater hammers (after IHC 1999)

pile drivability evolved from simply pounding the pile to refusal using the biggest available hammer to using mathematical models depicting the real-time pile-driving resistance.

The mathematical model concept is based upon the solution of the 1-D wave propagation equation. It allows the determination of the pile-driver adequacy and the soil's resistance to driving. The original development of the model wave equation concept began in the 1950s. In the 1960s, computer solutions to the 1-D wave equation were further detailed and refined. Applications to offshore piles (using above-water hammers) commenced in the 1960s (Bender et al. 1969) and led toward the analyses of different elements of the pile-driving process. Besides the hammer and the pile, accessories were analyzed during the actual pile-driving process. The contributions of pile leads (cradle), cushions, anvil, follow block (helmet or pile cap), and followers (chasers) to driving resistance were analyzed. The application of the wave equation to long piles unsupported through the water column was analyzed and found to accurately predict the behavior on several projects (Lowery, Edwards, and Finley 1969; Engeling 1974; Cunningham and Naughton 1977).

By incorporating the wave equation analysis during the early stages of the pile design process, preliminary selection of the pile hammer can be ascertained. During the pile-driving process, the wave equation analysis can be used to predict the pile's ultimate axial capacity (Hirsch, Koehler, and Sutton 1975; Porter and Ingram 1989). The wave propagation method was further refined to include computerized instrumentation, data acquisition, and data analysis thanks to electronic technology developments (Rausche, Goble, and Moses 1971).

Other wave propagation numerical solution techniques have also been proposed (Fischer 1975; Foo, Matlock, and Meyer 1977). During the 1980s, many case histories of pile-drivability analyses were conducted by the offshore industry, including hammer types (steam versus hydraulic) (Heerema 1980); hammer location (above-water versus submerged) (Aurora 1984); soil conditions (sands, clays, carbonate soils, weak rocks, layer conditions, etc.) (Agarwal, Rawat, and Paintal 1978; Tagaya et al. 1979; Aurora 1980; Stevens, Wiltsie, and Turton 1982; Stockard 1979, 1986); pile size (Lang 1980); and analysis methods (Holloway, Audibert, and Dover 1978; Van Zandwijk, van Dijk, and Heerema 1983). Additional computer codes that perform numerical integration of the wave equation were developed by offshore companies and universities. The university-developed codes include TTI, OCEANWAVE, TIDYWAVE, DIESEL1, WEAP, CAPWAP, DUKFOR, and PSI. The latest version of the WEAP series is GRLWEAP (Holloway, Audibert, and Dover 1978; Porter and Ingram 1989). Recent literature suggests that GRLWEAP accurately predicts pile drivability (Dutt et al. 1995, Doyle 1999).

Vibratory hammers. An alternate method of installing piles is pile advancement (driving) using vibratory methods. Vibratory hammer technology was developed by the Russians for onshore applications in the 1960s. The Japanese foundation industry quickly embraced this technology and developed an extensive array of vibratory hammers. The Americans became interested in vibratory hammers in the early 1970s.

Vibratory hammer power is supplied by electricity or hydraulics. The vibratory hammers that use hydraulic power normally use a diesel engine to drive the hydraulic pump, which in turn drives the excitation motor on the vibratory hammer. In the United States, hydraulic vibratory hammer systems have become dominant. The major manufacturers, including Vulcan, ICE, and MKT, offer hydraulic systems exclusively (Warrington 1989).

The vibratory hammer (one model shown in Figure 59) does not incrementally impart energy to the pile-soil system as does an impact hammer, but instead provides a rapidly alternating dynamic force to the pile. The hammers are designed to impart this dynamic force to the pile at a range of frequencies, i.e., from approximately 300 to about 7,200 rpm (5 to 120 Hz), depending on the type of pile to be driven. Frequencies above 30 Hz are considered to be "high frequency." These hammers generally provide for lowered transmission of ground excitation to neighboring structures. Hammers weighing greater than 12 tons with the capability of transferring dynamic force greater than 200 tons to the pile are available. Pile Buck, Inc., (1999) maintains a comprehensive list of manufacturers and equipment capabilities.

Case histories of offshore vibratory-driven piles are not as numerous as those for impact-driven piles, especially before the late 1980s. The evolution from above-water to underwater vibratory hammer usage occurred much faster than did the impact hammer evolution. It was soon realized that the benefits of underwater vibratory hammers outweighed the above-water benefits. Use of the underwater hammers began at offshore locations in the North Sea and Adriatic (Jonker 1987, 1988; Ligterink, Van Zandwijk, and Middendorp 1990; Ligterink and Martin 1992).

Pile drivability using a vibratory hammer is analyzed using the same 1-D wave propagation concept as for impact hammers. The model is somewhat different for vibratory hammers as compared to impact hammers, including modifications to wave equation computer programs. Chua, Gardner, and Lowery (1987) showed the applicability of the wave equation approach to vibratory hammer driving by modifying the TTI program (VIBEWAVE). Jonker (1988) adapted the TNOWAVE program to vibratory driving. Ligterink, Van Zandwijk, and Middendorp (1990) used the adapted TNOWAVE program to analyze the soil's resistance to vibratory driving during installation of 42-in. (1.07-m) piles. The vibratory pile-driving analyzer (VPDA) finite difference model was developed by Moulai-Khatir, O'Neill, and Vipulanandan (1994). The Wisconsin Vibratory Pile Driving Analyzer (WiscVPDA) was developed by Bosscher et al. (1998).

A list of considerations for selecting impact versus vibratory underwater hammers follows:

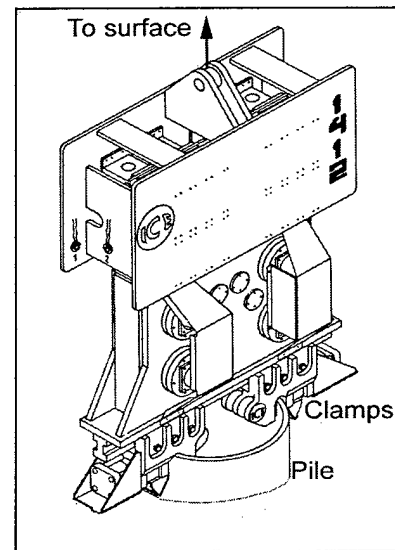


Figure 59. ICE 1412 hydraulic vibratory pile hammer (after Jonker 1987)

- a. No pile followers (chasers) are needed for underwater driving with either hammer.
- b. The vibratory hammer and pile are clamped together and are handled as one unit.
- c. The vibratory hammer can be activated prior to the pile tip touching the seabed. The methodology avoids the need for a temporary guide template (illustrated in Figure 60).
- d. The use of impact hammers is more common for offshore applications. Consequently, they have been used for a larger variety of soil types and pile sizes.
- e. Soil frictional resistance may be overestimated when using vibratory hammers.
- f. Vibratory hammers are used with piles requiring a minimum vertical tolerance, which allows easier realignment during penetration.
- g. A combination of hammers allows faster installation of a pile. The vibratory hammer can be used for initial positioning followed by limited driving to achieve lateral support, whereas the impact hammer can be used for the remainder of penetration.
- h. Above-water impact hammers generally have higher vibration and noise levels than vibratory hammers. Hydraulic impact hammers generally have lower vibration levels than diesel hammers.

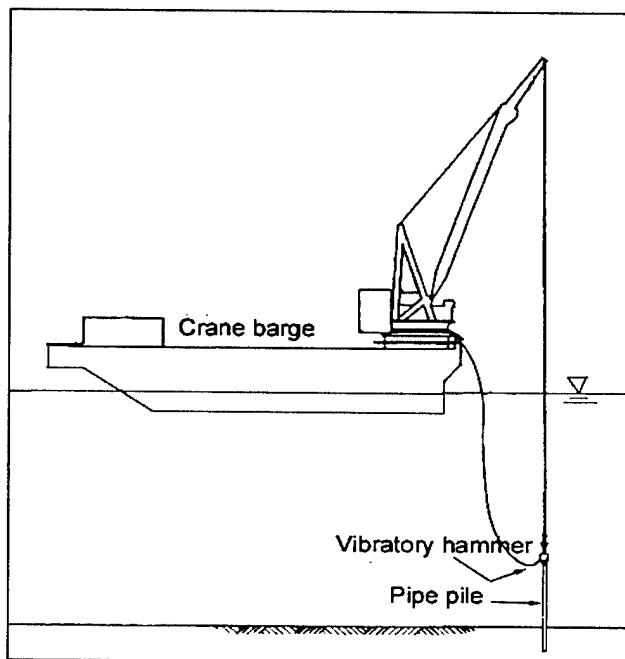


Figure 60. Vibratory-driven pipe pile without an underwater template (after Jonker 1988)

Technology is now available to reduce noise levels during above-water impact driving. Numerous manufacturers offer protective shields or encapsulation modules as noise mufflers. For example, one onshore contractor designed and built a hydraulic impact pile-driver that was about 12 db quieter than an equivalent system for driving concrete precast piles near an urban development. Totally enclosing all moving parts and eliminating all metal-to-metal contact reduced the noise level to the point at which the ropes flapping against the rig's mast was the only appreciable noise (*Ground Engineering* 1999). For driving tubular and standard steel sheet piles, a Japanese company has recently demonstrated in the

United States a hydraulic pile-driving system that operates at less than 60 db of noise and is virtually vibration free (Phair 2000).

Drilled shafts. Drilled shafts (also called drilled piers, drilled caissons, large-diameter bored piles, or rock sockets) can be installed by a variety of onshore drilling rigs. Offshore equipment is generally the same with the exception of being mounted on a barge or other floating platform. Casing is driven or drilled past the mudline. If the casing can be sealed into an impervious stratum, it can then be dewatered and drilled. Otherwise, drilling is conducted through casing using the mud-drilling technique. Cuttings are removed using air or water ejectors or by mud circulation techniques. After cleanout and inspection, steel reinforcing and concrete are then placed into the casing to form the shaft (Greer and Gardner 1986).

A more common offshore method that does not use reinforced concrete is typically used on steel jacket platforms sited on rock regardless of whether soil overburden is present. Instead, a steel pipe pile is lowered into the bored or cased opening, and then the annulus is grouted to provide a "pinned" pile system. When casing is used, it is advanced by spinning the casing with a drill bit attached at its lower end or by guiding a smaller diameter casing through a jacket pipe sleeve or spud pile and then advancing the casing by use of a pilot drill bit. Grouting of the casing may or may not be conducted prior to insertion of the inner pipe pile. Final grouting is accomplished after the inner pipe pile is advanced to its target depth. The drill rig may be operated from a support barge or may operate directly on top of the outer casing.

Steinke and Strasser (1978) discussed drilling techniques for the Valdez terminal where piles up to 3.5 ft (1.1 m) in diameter were placed in 43-ft (13-m)-deep boreholes located up to 1,500 ft (456 m) from shore, on steep 60-deg underwater rock slopes. Fox (1970) discussed rotary drilling through driven 40-in. (101-cm)-diam steel pipe piles with an 18-in. (46-cm) tri-cone bit on a 7-in. (18-cm)-diam drill stem to clear out boulder clay overburden prior to rock drilling. Drill water pumped at 2,100 gpm (8,400 L/m) helped to clean the soil cuttings as the bit advanced into the rock, although boulder inclusions caused problems during the drilling operations. After achieving the design depth, each pile was then filled with reinforcing bars and concrete. Chamberlin (1970) discussed drilling 42-in. (107-cm)-diam holes in layered limestone using a 24-in. (61-cm) tri-cone bit, a 32-in. (81-cm) hole opener, and a 42-in. (107-cm) underreamer (shown in Figure 61). A 36-in. (91-cm)-diam pipe pile was then lowered into each socket during drilling and was grouted inside and out. Santiago et al. (1986) discussed drilling techniques used at an offshore platform in 350-ft

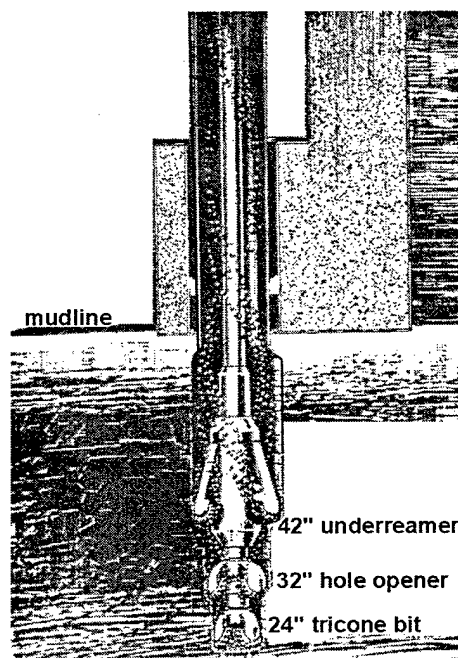


Figure 61. Offshore rotary drilling operation for grouted pipe piles in limestone (after Chamberlin 1970)

(106-m) water depth to install 4.5-ft (1.37-m)-diam pipe piles 82 ft (25 m) into claystone and gravel deposits.

Suction caissons. Suction caissons (also called suction piles, buckets, skirts, skirted foundations, or anchors) are installed by self penetration into the seabed, similar to the illustration shown in Figure 62.

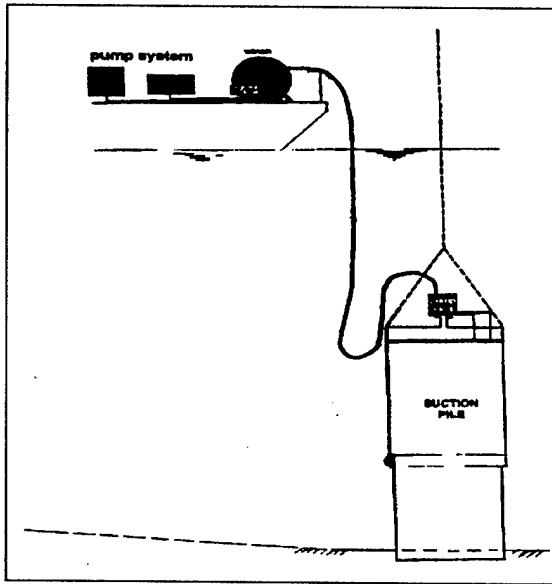


Figure 62. Suction caisson (pile) installation (after Senpere and Auvergne 1982)

The initial phase is self-weight penetration as the caisson is lowered onto the seabed and allowed to “sink” under its own weight into the mudline. Further penetration requires an underpressure in the cavity of the suction caisson, which creates an additional driving force. As underpressure is applied (usually by a skid-mounted pump on top of the caisson, shown in Figure 63), the caisson tends to penetrate the soil until the pressure inside the skirt walls tends to equalize with the external pressures. The rate at which the pore pressures tend to equalize and mechanisms by which the underlying soil tends to fail are due to the material and strength properties of the soil. Set-up effects (time-dependent properties) are similar to those found in pile driving (Andersen and Jostad 1999).

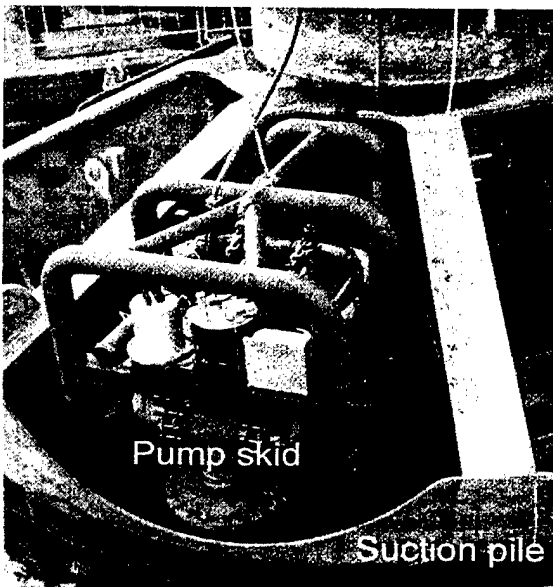


Figure 63. Pile-mounted suction pump skid (after Suction Pile Technology, Inc. 1999)

Placement positioning. Proper 3-D positioning of the foundation is required during the installation process. The installation procedures must be consistent with the design criteria, and advance planning for alternate or supplemental procedures should be made prior to mobilization of the marine construction equipment (Sullivan and Ehlers 1972). Tanaka et al. (1987) studied the installation procedures for placing a gravity-based foundation on an offshore Japan seabed. Model testing and numerical simulation confirmed the technical feasibility and provided recommendations prior to actual installation.

For typical offshore oil and gas structures, foundation positioning is intricately linked to the exploration and drilling process. Elaborate computerized

positioning procedures are followed, which have been discussed in the offshore literature for the largest, most complex, or deepest structures from subsea risers to surface platforms.

For example, positioning and installation procedures for Shell's Cognac jacket platform are detailed by Collipp and Johnson (1979), Simpson (1979), Mayfield, Strohbeck, and Wilkins (1979), and Sterling et al. (1979). Since the foundation pile-driving guide or "template" was integral to the jacket structure, no positioning of a separate pile-driving template was required. During the positioning and placement of the 84-in. (2.1-m)-diam by 615-ft (187-m) pipe piles, an acoustic positioning system composed of multiple transponders (beacons and hydrophones) communicated with a computerized referencing system. Hydrophones were mounted on the pile-lowering system, which received sequenced pulses from oceanfloor and surface-barge beacons. Three-dimensional computer-assisted pile-maneuvering was performed to guide the platform jacket stab guides onto the piles. A pile-mounted TV camera system and a video ROV provided video monitoring capabilities. After pile positioning and stabbing were completed, the pile hammer monitoring and control system was implemented. Three operators were positioned at separate computer/TV consoles to control and monitor the pile elevator depth, the hammer depth, and the hammer operation. Although the installation schedule allowed 3 days per pile, the installation of the final 18 piles (out of 24 total) required a total of only 22 days.

Pile-driving templates that are not preattached to the structure require separate positioning and placement efforts. Ulbricht et al. (1994) discuss positioning efforts for Shell's Auger Tension Leg Platform (TLP) foundation template, which is shown in Figure 64. The foundation template was placed at a depth of 2,860-ft (870 m) in the Gulf of Mexico to within tolerances of 2 ft radial offset, 2 deg heading offset, and 2 deg horizontal offset.

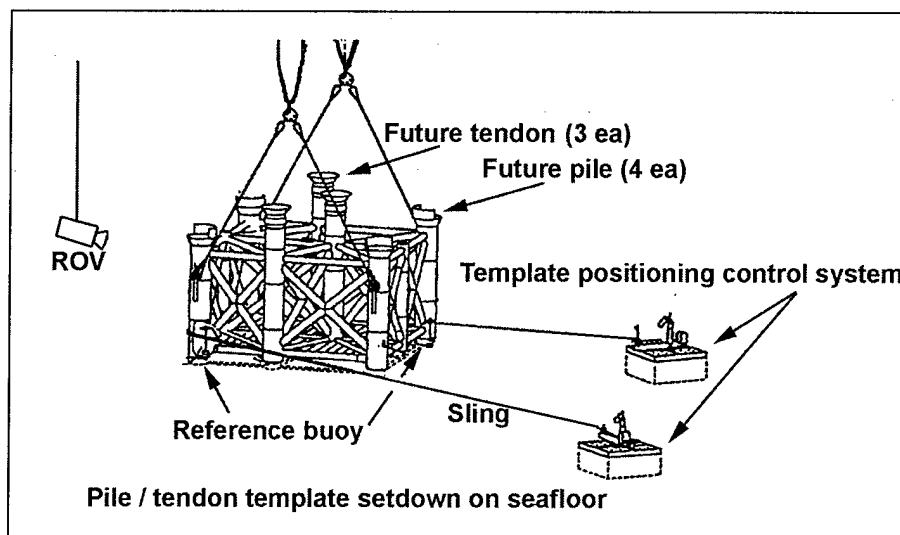


Figure 64. Pile template positioning for Shell's Auger TLP (after Ulbricht et al. 1994)

The installation vessel (a semisubmersible crane vessel) experienced low-frequency surface excursions between 15 and 35 ft (5 and 11 m) while setting the 60- by 60- by 48-ft (18- by 18- by 15-m), 611-ton steel template on a slightly sloping, soft clay site. Specially designed "clump weights" were acoustically positioned on the seabed prior to lowering the template. Near- and far-field acoustic transponder arrays on the seabed provided positional references to the template-mounted Sonardyne MicroNav acoustic monitoring system. A dual-axis inclinometer was attached to the template and was acoustically monitored during lowering. Template position, heading, inclination, and water depths were monitored relative to the crane vessel's position. As the template neared the seabed, ROV-connected slings attached to the "clump weights" corrected and stabilized its position prior to setdown. Pile stabbing and hammer operations were monitored using acoustic positioning equipment.

Offshore pile-handling operations have improved as more experience has been gained in deepwater pile transporting, offloading, upending, and lowering. Converse et al. (1990) discussed a newer, more mechanized method for handling large, one-piece piles while reducing the requirement for specialized rigging.

With experience, positioning tolerances have also become tighter. In 1979, the accuracy using the computerized positioning system was within a radius of about 15 ft (4.5 m) at a depth of 1,000 ft (304 m) for Shell's Cognac platform. By 1999, piles were positioned within 1 ft (0.3 m) of the target with less than 1 deg of final tilt in 3,800-ft (1,160-m)-deep water for the Shell Mars, Ram-Powell, and Ursa TLPs (Doyle 1999).

For positioning a drilled shaft casing, differential GPS (DGPS) surveying was demonstrated at the Corp's Lock and Dam 24 on the Mississippi River. Rotary-drilled casings were monitored for DGPS positional accuracy and compared to standard surveying methods (U.S. Army ERDC 1999).

Foundation-structure connections

The design and installation of mating connections between an underwater foundation and its supported structure are not within the scope of this study and, hence, are not specifically addressed in this report. For the improved-site foundation, there usually is no distinct connection to the structure. Gravity-based and pinned foundations typically require connection to their structures, and their installation must be considered during the design phase.

Various connection configurations are available, including shear pins, stabbed or embedded mating connections, and grouted connections. Some connection details are included in offshore design guidance (American Petroleum Institute 1993). Several offshore examples include the discussion of mating details and installation procedures for jacket structures on piles by Allen et al. (1990); a procedure to mechanically connect piles to an underwater structure using hydraulic swaging equipment by Ulbricht et al. (1994); the use of pile stabbing guides by Birdwell and Jordan (1994); and a Smart Leg system by

Labbé, Legras, and Standing (1999) for float-over installations on piles. Sele and Skjold (1993) and Welham and Gilfrin (1993) discuss grouted connections.

Underwater Foundation Testing

Underwater foundation tests may be of two types: integrity testing and performance testing. Integrity testing seeks to determine the quality of the constructed foundation material, i.e., to determine if the concrete in the drilled shaft is honeycombed. Performance testing seeks to determine the adequacy of the foundation to withstand the design loading (i.e., the expected range of deflections imposed by lateral loading on a drilled shaft) and the capacity of the soil to resist the shaft (or pile) loads.

The type and degree of testing usually depends upon the type of foundation and economics. The general trends for underwater foundation site testing are listed below.

Foundation	Integrity Tests	Performance Tests
Improved-site	Material-dependent	Intrusive (sampling, etc.) and NDT
Gravity-based	NDT	Long-term monitoring
Pinned	NDT	NDT, large-strain tests or load tests

Intrusive or destructive tests commonly refer to invasive techniques such as soil/rock coring, sample retrieval and laboratory tests, or in situ techniques such as cone penetrometer and shear vane tests. These are the same tests that are used in geotechnical site investigations to determine the soil/rock properties including shear strength, friction angle, relative density, RQD, etc. Intrusive testing may also be required in gravity-based and pinned foundations to determine strength of (cored) concrete specimens.

Nondestructive tests (NDT) cover a broad range of techniques. Integrity testing using NDT is primarily concerned with cast-in-place concrete or grout materials found in gravity-based and pinned foundations. Flaws in steel and timber piles may also be detected using NDT during or after pile-driving. Performance testing of driven piles to determine their drivability and ultimate capacity may also be accomplished with NDT methods. According to Chernauskas and Paikowsky (1999), the NDT methods of integrity testing are categorized as small-strain testing and large-strain testing.

Small-strain integrity testing

Small-strain test methods generate stress waves that are interpreted to provide shaft or pile integrity information, such as concrete or grout integrity. Small-strain integrity testing includes direct transmission techniques, including crosshole sonic logging (CSL), single-hole sonic logging (SSL), and parallel

seismic logging, as well as surface reflection techniques, such as pulse echo method (or sonic echo), transient dynamic response (impulse response), and electromagnetic techniques (electromagnetic induction (EM), X-ray, radar, and magnetometer).

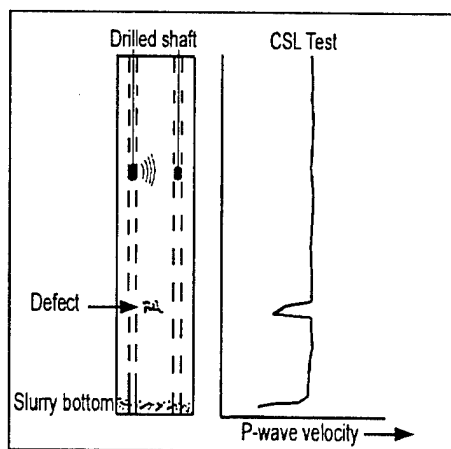


Figure 65. Crosshole sonic logging (CSL) integrity test

The most common integrity testing technique is the CSL, which is conducted within preinstalled tubes in cast-in-place piles or drilled shafts (illustrated in Figure 65). A transducer is lowered into one water-filled tube and emits an acoustic signal that is received by a hydrophone (geophone) transducer in a separate tube. The signal's compression (P-) wave is detected and compared to the transit time of a signal transmitted through intact material with a given modulus of elasticity. If material defects are present, the wave speed will be reduced relative to that of intact material. Accurate transducer positioning helps to pinpoint the location of a defect. For accurate P-wave transmission, the concrete material must be fully cured. Chernauskas and Paikowsky (1999) describe the first use of a portable personal computer-based CSL test on concrete drilled shafts in the United States with the Pile Integrity Sonic Analyzer software package.

Single-hole sonic logging uses only one tube in the drilled shaft. This method is normally used after construction is completed and it has been determined that there is a need to check the shaft integrity by coring a single hole through the shaft.

Parallel seismic logging (as illustrated in Figure 66) is similar to geophysical cross-hole seismic testing in that holes are drilled into the surrounding soil. A

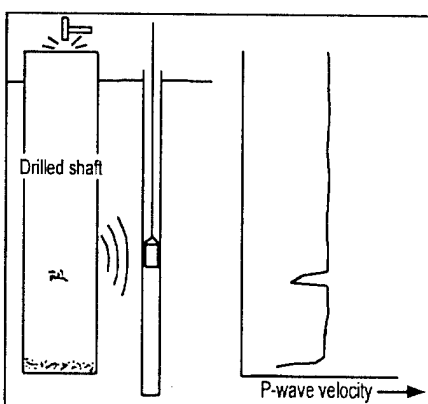


Figure 66. Parallel seismic integrity test

PVC (polyvinyl chloride) casing is inserted into each hole, the annulus is grouted, and the casing is filled with water. Both SSL and parallel logging use equipment and interpretation procedures similar to those used in the CSL test.

The pulse echo method (PEM) requires an external hammer source and an accelerometer transducer mounted on the drilled shaft (or pile), as shown in Figure 67. The hammer impact causes small strains, which induce stress waves in the shaft that are transmitted to the accelerometer. The accelerations are integrated with respect to time, to obtain velocity profiled as a function of shaft length. The presence and location of defects are interpreted from the velocity record. The effectiveness of this technique is limited to depths up to 20 to 30 pile diameters.

The transient dynamic response (TDR) method (not to be confused with time domain reflectometry) requires an external instrumented hammer. The impact force of the hammer is recorded and combined with the acceleration record, similar to the PEM method. The shaft's velocity response due to the induced excitation force can be interpreted to indicate the presence and location of defects.

In general, electromagnetic techniques are not commonly applied in pile integrity testing. However, two manufacturers of equipment and software specifically designed for pile integrity testing using surface reflection methods (low strain) are PileTestCom (Israel) Ltd.'s Pile Integrity Sonic Analyzer and Pile Dynamics (U.S.A.) Inc.'s Pile Integrity Tester.

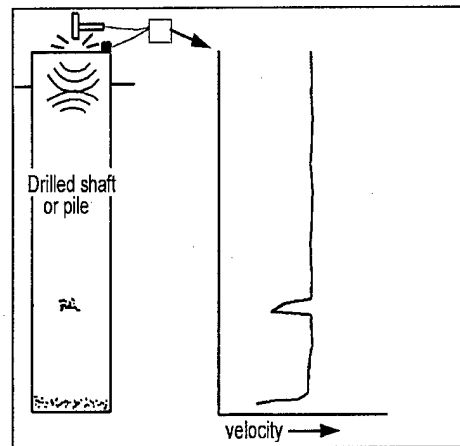


Figure 67. Pulse echo method integrity test

Large-strain testing

Large-strain testing is typical of the integrity/performance testing conducted during pile driving. Large strains induced during pile driving generate large stress waves that mobilize the shaft or pile resistance against the surrounding soil. Large-strain methods (also referred to as "dynamic testing") allow interpretation of both material integrity and performance. The ability to determine the shaft or pile performance allows one to evaluate drivability as well as load carrying capacity. One example of a large strain test is the surface reflection method, which is similar to the pulse echo and transient dynamic response methods.

Large-strain methods are primarily used for driven (impact and vibratory hammered) piles. When the hammer strikes (or vibrates) the pile head, a large strain wave is initiated and propagates down the pile. External soil resistance or changes in the pile's impedance (due to variations in the material or geometry of the pile) initiate reflection waves that are recorded.

Typical dynamic pile testing instrumentation requires two accelerometers and two strain transducers mounted on opposite sides near the pile top. The strains are converted to forces according to the theory of elasticity while the accelerations are converted to velocities. Although numerous interpretation methods are available to assess the pile's integrity and performance characteristics, all are generally based upon 1-D stress wave propagation theory. The U.S. standard for high-strain dynamic testing of piles is ASTM Standard D4945 (ASTM 1993). One of the most common wave equation software packages is GRLWEAP for Windows. A U.S. company that offers it, as well as other proprietary pile testing systems, is Pile Dynamics, Inc., of Cleveland, OH. The PDA (Pile Driving Analyzer) and the PIT (Pile Integrity Tester) systems are also sold by this company.

The PDA systems (PAK and PAL) monitor acceleration and strain sensors attached to the impact-hammered pile and process these signals (using GRLWEAP) after each blow of the hammer during driving or restriking to investigate hammer performance, driving stresses, and pile integrity and to provide an estimate of static bearing capacity (Pile Dynamics, Inc. (PDI) 1999).

The PAK collects up to four channels of strain and four channels of acceleration (two piezoelectric and two piezoresistive). Compared with PAL, which is a smaller unit that can capture only two channels of data for each of strain and acceleration, PAK has superior data collection and processing capabilities. Remote data collection using a cell phone modem is also possible, and underwater transducers are available. Figure 68 illustrates a PDA test example.

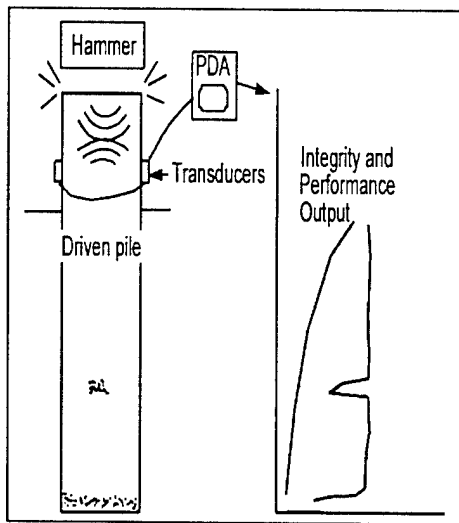


Figure 68. Pile dynamic analysis (PDA) test

Using the software program CAPWAP (Case Pile Wave Analysis Program), which was developed by Goble Rausche Likins and Associates, Inc. (GRL), and marketed by PDI, Inc., the soil response is expressed in terms of total capacity and its distribution along the shaft and at the toe. Damping factors and quakes are also modeled. CAPWAP users are required to have approved training. GRL and PDI regularly offer seminars and workshops.

Performance testing of vibratory-driven piles is generally based upon the same principles as large-strain dynamic testing of impact-driven piles. The impact-hammer equations are modified to account for the dynamic properties of the vibrating energy source. Computer models based upon finite difference or finite element methods for solution of the wave equation have been proposed and are similar to those models previously developed for impact-driven piles.

The computer programs VIBEWAVE and TNOWAVE were developed to predict pile-driving performance and the soil's resistance to vibratory driving. TNOWAVE can also be used to analyze the performance of the vibratory hammer as well as to predict several parameters, including maximum stress in the pile, amplitude of vibration at different levels in the pile, penetration rate, and nearby building vibrations. TNOWAVE is especially useful for matching vibratory hammer size to the soil conditions and preventing pile damage due to overloading. The computer model VPDA was developed to predict the pile bearing capacity in addition to predicting the pile-driving performance (Moulai-Khatir, O'Neill, and Vipulanandan 1994). The predicted bearing capacity is input into the program and compared to the observed penetration rate. When model parameters match, the actual bearing capacity of the pile is calculated.

Bosscher et al. (1998) used VPDA and field (onshore) load tests to develop the computer model WiscVPDA, which estimates the ultimate bearing capacity

as the pile is being driven. WiscVPDA was developed and refined by comparing the predicted (VPDA) bearing capacity to the full-scale load test ultimate bearing capacity results (illustrated in Figure 69). Steel pipe piles and H-piles at one chosen site were vibratory driven and load tested for statistical correlations. Significant correlations were noted for pile parameters such as type, diameter, perimeter, and cross-sectional area. Significant vibratory hammer correlations, i.e., driver parameters such as driving frequency, eccentric acceleration, and delivered power, were noted and incorporated into the program.

Load testing

Static load tests are conducted to determine the shaft (or pile) performance and the soil response in resisting applied axial and lateral loads. A load test is usually made for one or more of the following reasons (Whitaker 1976): (a) to obtain a load-settlement or load-displacement relationship, (b) to obtain or validate the design safety factor against failure, and (c) to determine the true ultimate bearing capacity as a check on the design value.

Pile load test configurations are axial (compression, tension, and quick load), monotonic lateral, and cyclic lateral. These test configurations are detailed in ASTM Standard D 1143 (axial compression and quick load), ASTM Standard D 3689 (axial tension), and ASTM Standard D 3966 (lateral loading) (ASTM 1993).

The quick load test is permitted as a faster alternative. Its two loading configurations are the constant-rate-of-penetration (CRP) and the quick maintained-load test (ASCE 1993). The CRP test was developed in the United Kingdom by Whitaker (1976) and is not as popular in the United States as is the maintained-load test (ASCE 1993). In the maintained-load test, the load is applied in increments of 10 percent of the proposed design load and is maintained for a constant time interval of a few minutes. In the CRP test, the load is applied to cause pile head settlement at a predetermined constant rate, usually 0.01 to 0.1 in./minute. Several static bearing capacity determination methods (including the Corps of Engineers method) are listed in ASCE (1993).

Axial pile load tests conducted in the past used superimposed dead weights. Current practice includes the use of a hydraulic ram that is jacked against a loaded platform or against a test frame anchored by reaction piles (ASCE 1993).

Load tests conducted onshore are much more numerous and are relatively less costly than those conducted offshore. The offshore industry literature contains several references to offshore load tests, especially for research purposes

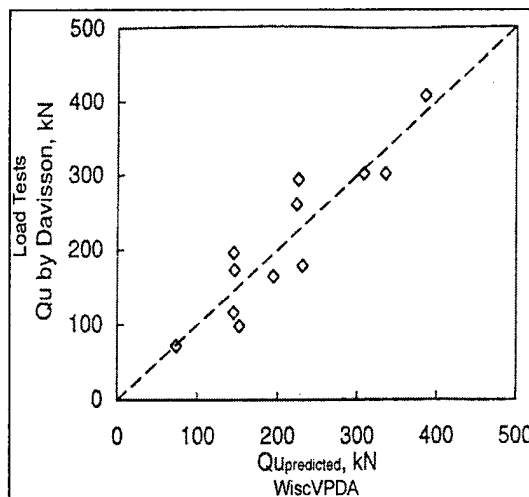


Figure 69. Comparison of actual pile load test results to WiscVPDA predictions (after Bosscher et al. 1998)

or in riskier environments such as calcareous soils. For example, load tests conducted from both a fixed offshore platform and a floating vessel are described by Angemeer, Carlson, and Klick (1973) and Angemeer et al. (1975) for offshore Australian sites containing calcareous soils. Dutt and Cheng (1984) conducted pullout load tests on 2-ft (0.66-m)-diam pipe piles in offshore carbonate sands. A research program described by Matlock, Bogard, and Chan (1998) conducted load tests on pipe piles from a fixed platform in the Gulf of Mexico. Jardine and Overy (1996) conducted load tests on 2-ft (0.66-m)-diam pipe piles in dense sand from a fixed platform in the North Sea.

The majority of referenced offshore load tests are performed under axial tension loading (monotonic pullout tests) to enable determination of skin friction capacity. An offshore load test for bending, tension, and compression of pipe piles driven into carbonate bedrock was detailed by Settgaest (1980). Vines and Hong (1984) reported tests on large-diameter laterally loaded pipe piles in coral at an offshore port site in Saudi Arabia.

Load tests conducted onshore with the results applied to offshore structures have also been conducted. Tension and compression loading tests were conducted on 2-ft (0.66-m)-diam pipe piles in stiff clay in Belgium (Heerema 1979). Grosch and Reese (1980) performed cyclic axial loading tests on small-scale (1-in.-diam) piles pushed beyond the end of shallow boreholes in soft clay. Lu (1986) evaluated small-scale pile models in carbonate sands while Karlsrud and Nadim (1990) reported tests on small-scale (6-in.-diam) piles subjected to tension and compression loadings in clay. Zuidberg and Vergobbi (1996) discussed load tests on 30-in. (0.76-m)-diam pipe piles in dense silica sand for the European Initiative on Piles in Dense Sands (EURIPIDES) Project. Onshore testing of pipe piles driven and grouted in calcareous soils was performed by Randolph et al. (1996). Onshore axial load tests on 4-ft (1.2-m)-diam pipe piles in clay were conducted for the Corp's future in-the-wet IHNC project in New Orleans (U.S. Army Corps of Engineers 1999a).

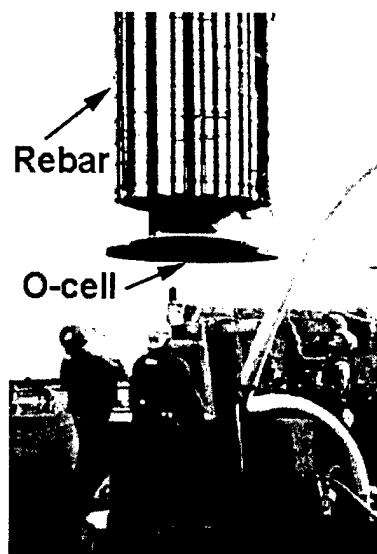


Figure 70. Osterberg cell at the rebar cage bottom (after DiMillio 1999)

Load tests conducted on drilled shafts sometimes use the Osterberg load cell (Figure 70). As compared to conventional load tests conducted with a reaction load or hold-down shafts with a heavy reaction beam, the load test employing the Osterberg load cell is relatively easy and inexpensive (Greer and Gardner 1986).

The Osterberg method, which was developed in 1984, allows separate measurements of end-bearing and friction loads versus deflection for each load increment.

Typically, a flat pressure cell that covers the bottom of the shaft hole is hydraulically pressed simultaneously against both the bottom of the shaft hole and the bottom of the shaft. Depending upon the cell placement within the drilled shaft, gauges can be used to measure the cell's downward movement

against the soil and its upward movement against the shaft. From these data, load-deflection curves are generated. The most common loading sequence is the ASTM Quick Test Method D 1143 (Osterberg 1998). After the testing is completed, the cell can be grouted in place, and the test shaft can be used as a production shaft.

Drilled shaft (caisson) load tests were conducted in 1998 at the Corp's Monongahela River Braddock Dam site to provide axial and lateral load data for design of the production shafts (Figure 71).

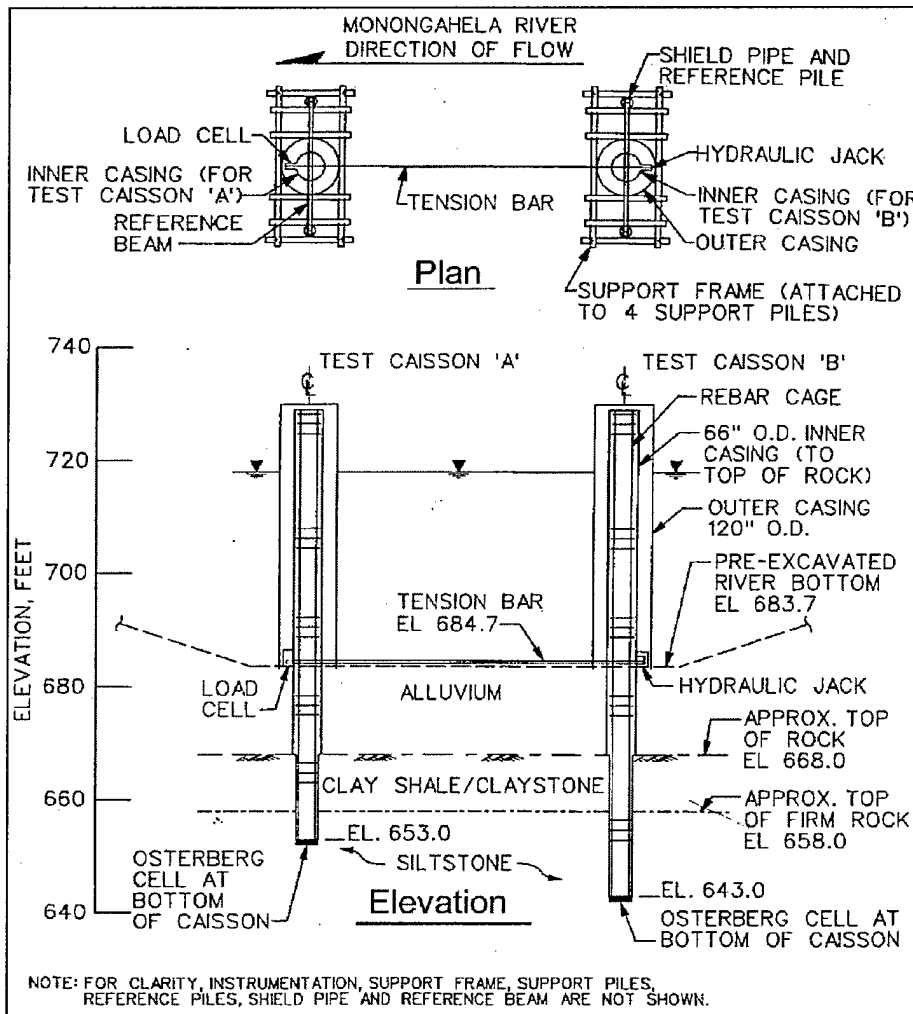


Figure 71. Braddock Locks and Dam 2 drilled shaft load test configuration (after Wahl 1999)

Two 5-ft (1.5-m)-diam concrete-reinforced shafts were embedded 15 and 25 ft (5.1 and 7.6 m), respectively, into the claystone/siltstone bedrock. To the rebar cages for each 70-ft (21-m)-long shaft, strain gauges, telltale tubes, CSL tubes, inclinometer tubes, and an Osterberg cell (at its bottom) were attached. The rebar cage was lowered through the casing, and then the casing was

backfilled with tremied concrete. After the concrete had cured, integrity and performance tests were conducted. Integrity testing was conducted using CSL. Performance testing was conducted using the Osterberg cell for application of axial loads and hydraulic jacks for application of lateral loads. Axial load-deflection curves were generated to provide end-bearing and side-wall friction resistance design data, while bending moments were generated from the lateral load-deflection curves (Wahl 1999).

Full-scale load tests of underwater drilled shafts have also been conducted at the Ohio River (Kentucky), St. Mary's River (Georgia), and the Apalachicola River (Florida) bridges using the Osterberg cell (Osterberg 1998). Load tests on shafts up to 9 ft (2.7 m) in diameter were conducted in water depths to 27 ft (8 m).

Over 300 load tests using Osterberg cells have been conducted in over 10 countries. Drilled shafts (bored piles or caissons), driven pipe piles, and driven precast concrete piles up to 10 ft (3 m) in diameter, driven to depths up to 300 ft (90 m), have been tested. Loads up to 15,000 tons (135 MN) have been applied using Osterberg cells. Osterberg (1998) lists several installation methods and cell locations for determining side shear resistance and end-bearing values. As an example, Figure 72 shows the test configuration on a drilled shaft subjected to deep scouring.

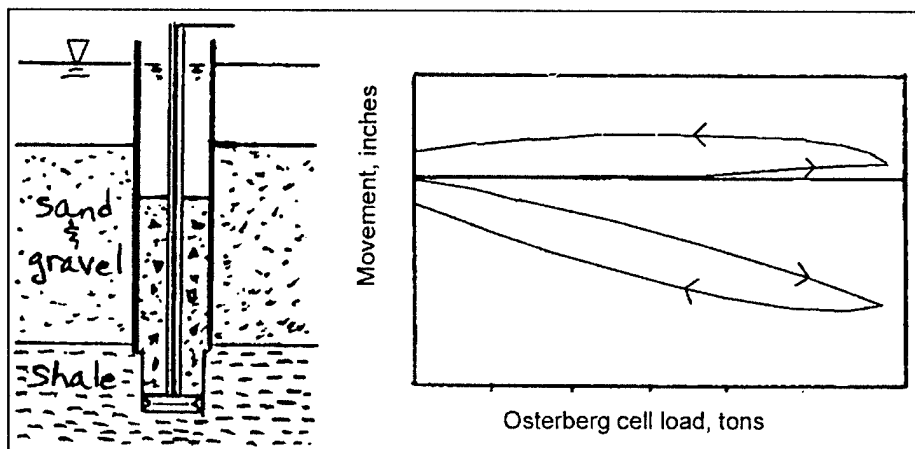


Figure 72. Load test on an underwater drilled shaft showing a test configuration for scour design with load/deflection curve results (after Osterberg 1998)

Long-Term Considerations

Underwater foundations are designed and constructed for durability and long-term integrity during their service life. After construction is completed, important considerations should include protecting the foundation from scour, corrosion, and deleterious environmental effects and ensuring that the foundation performs as designed.

Deleterious effects

Constructed foundations are exposed to swift currents, shifting mudline soils, corrosion, and other (harmful) environmental conditions. Each foundation type is susceptible to any of these conditions to a varying degree.

For example, an improved-site foundation is susceptible to scouring, but corrosion is generally not a consideration. A steel pipe pile foundation exposed above the mudline will be more susceptible to corrosion than a submerged concrete gravity-based foundation. A harmful effect that does not discriminate by foundation materials or foundation types is mudline erosion (scouring), as illustrated in Figure 73.

Scouring. Scouring or erosion is caused by unidirectional water currents, oscillatory waves, and tidal currents. Scouring is the result of the interaction between the water flow field, the obstruction to this flow field (the foundation), and the sediment bed (Machemehl and Abad 1975). The scouring phenomenon has been researched for different foundation types, and several technological advances have been developed to reduce scouring action.

Ninomiya, Tagaya, and Murase (1972) studied the scouring of gravity-based foundations (cylindrical shapes) placed on soft cohesive and sandy soils. Model testing using square foundation shapes resulted in formulas for scouring depth and pattern due to currents. Correlations with other foundation geometry shapes were also made. It was concluded that a skirted hemispheric shape was the most effective shape to resist scouring. Scouring characteristics were also quantified.

For pile foundations, Abad and Machemehl (1974) and Machemehl and Abad (1975) researched the effects of scour caused by oscillatory wave motion and unidirectional currents in a laboratory wave flume. The parameter volumetric scour, rather than scour depth, was studied based upon the controlling factors, which included current velocity, water depth, wave characteristics, and pile diameter. These researchers were able to predict the scour patterns and the magnitude of the scour when the geometric and dynamic similarities of a given foundation were matched to the research results.

Chow and Herbich (1978) studied sand scour due to oscillatory wave motions around a pile group in a laboratory wave flume. Fifteen variables controlling the ultimate scour depth were analyzed. The variables were water depth, density, viscosity, and free stream velocity; wave height, period, and length; sand density, diameter, and friction angle; elapsed time and gravity acceleration; pile diameter and distance between piles; and ultimate scour depth.

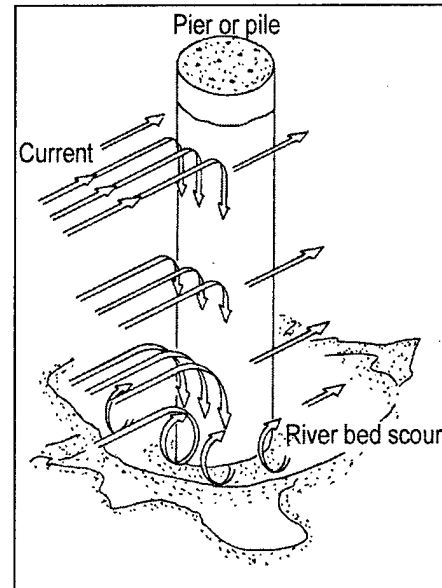


Figure 73. Scour development around a pinned foundation (after Abad and Machemehl 1974)

Maidl and Schiller (1979) conducted field research at a gravity-based platform in the North Sea to investigate scour phenomena and protection techniques. The protection techniques included (a) precast slabs hinged to the gravity-based foundation, (b) sandbags in nylon netting, (c) nylon mats filled with colcrete grout, and (d) mats studded with artificial seaweed. Costs, installation techniques, and recommendations were discussed for the first three techniques. It was concluded that the sandbags in nylon netting had the lowest service life and efficiency rating, while the nylon bags filled with colcrete rated the highest.

Sheppard, Niedoroda, and Karanumuni (1990) compiled all previously researched scour phenomena and synthesized the data. These researchers compiled 98 data sets from five investigations of steady flow around vertical cylinders in cohesionless granular soil. They proposed an equation to predict the erosion depth in a scour hole formed by a steady current passing a vertical cylinder.

Scour induced during pipeline installation was addressed by Bijker et al. (1991). Scour induced during cyclic lateral loading of piles was described by Reese, Wang, and Long (1989).

The installation of scour protection consists of side- or fall-pipe dumping, or individually placing selected fill material (rocks and gravel). The most common offshore methods are side-dumping and fall-pipe placement, largely due to the massive quantities of materials that are required around some of the platform foundations (Figure 74) and for seafloor trenched pipelines. For precision placement, the fall-pipe method is most frequently used.

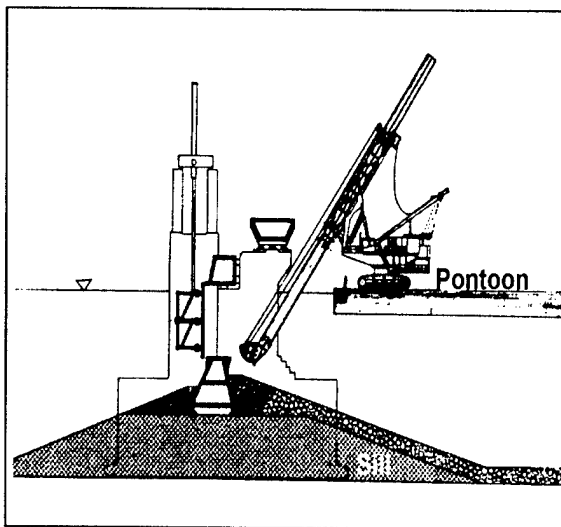


Figure 74. Offshore fall-pipe rock dumping (after Tideway BV 1999)

Specialized vessels are available to place offshore scour protection. For example, Tideway BV (Dredging International NV, Netherlands) has a dynamically positioned (no tug assistance needed) fall-pipe vessel capable of placing 12,000 tons of rock within a 1,644-ft (500-m) zone to a depth of 986 ft (300 m). Its survey system includes a multibeam echosounder and an ROV that is heave-compensated to within 4 in. (10 cm) vertical movement. Other large offshore vessels with rock-dumping capabilities are owned by HAM (Hollandsche Beton Groep NV, Netherlands), Ballast Nedam NV (Netherlands), and Boskalis Offshore BV (Netherlands).

Precision-placement may also be accomplished by individual rock placement. A specially adapted barge (pontoon) crane deposited rocks weighing

more than 1 ton on the Oosterschelde project (shown in Figure 75). Basalt rocks weighing 5 to 9 tons (6 to 10 metric tons) were placed as the top layer of scour protection on the submerged pier sills. Rocks weighing less than 1 ton were placed by side-dumping vessels (DOSBOUW 1987).

Corrosion. Corrosion is caused by numerous environmental factors. Its prevention is an important topic, especially to the metallurgical industry. Substantial research has been conducted regarding marine corrosion and fouling, and several organizations, including the National Association of Corrosion Engineers, American Society of Mechanical Engineers, and American Concrete Institute, and publications including *Corrosion* and *Materials Performance* have addressed these issues. Steel pilings are especially susceptible to corrosion. For example, the average general corrosion rate of marine structures in European countries is approximately 5 mils (0.125 mm) per year (Van Damme and Vrelust 1999).

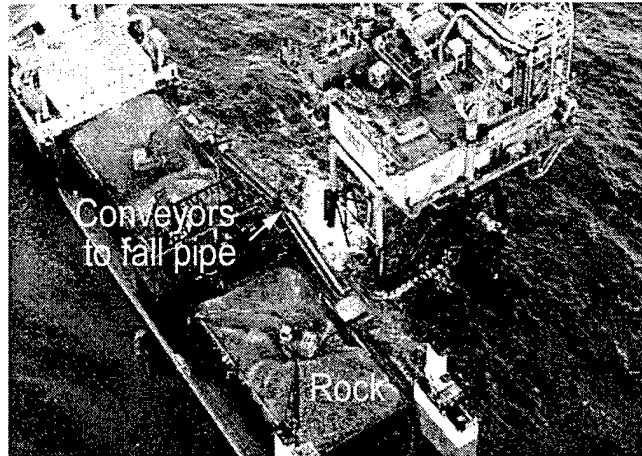


Figure 75. Precision rock placement (after DOSBOUW 1987)

As illustrated in Figure 76, the corrosion and biological fouling exposure zones on a typical pile include the lower erosion zone, the biological fouling zone, the splash zone, and the atmospheric zone (Escalante and Iverson 1978).

Although each zone has a different corrosion profile, the highest corrosion rate occurs in a narrow band just below the splash zone and is aggravated by fluctuating water levels (Van Damme and Vrelust 1999).

Environmental factors influencing corrosion include dissolved oxygen, temperature, salinity, pH, currents, pressure, and biological fouling (Thomason and Fischer 1991). Corrosion prevention methods for steel piles include barrier type coatings, anodic coatings, cathodic protection, or combinations of these (Escalante and Iverson 1978).

Escalante et al. (1977) evaluated a variety of coatings and cathodic protection systems on steel piles located off the Atlantic coast over a 15-year period. Van Damme and

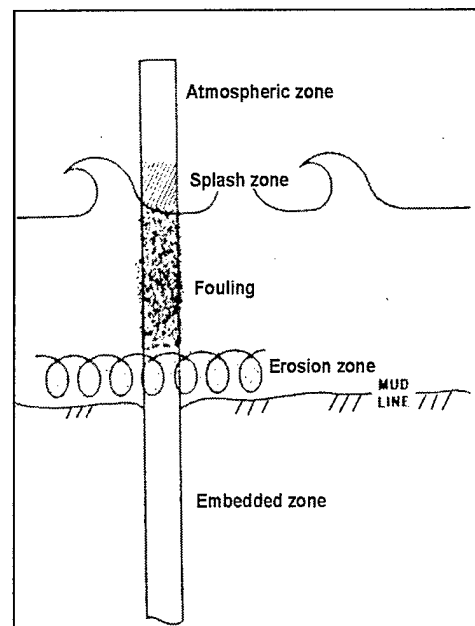


Figure 76. Exposure zones for piling (after Escalante and Iverson 1978)

Vrelust (1999) investigated the effects of a two-component coal tar and epoxy coating system over a 15-year period.

Cathodic protection is needed for long-term corrosion protection of subsea components. The offshore industry has successfully used cathodic protection measures over the last several decades, and reliable systems have been optimized for water depths less than 986 ft (300 m). For example, North Sea experience has shown that inducing a high current density (e.g., 320 mA/sq m) on steel piles immediately upon immersion promotes rapid cathodic polarization and formation of high-quality calcareous deposits. The calcareous deposits reduce the dissolved oxygen diffusion rate and thus reduce the long-term current density requirements for cathodic polarization (Thomason and Fischer 1991).

Combination cathodic protection and multicoat paint systems offer greater corrosion resistance. Shell's Gulf of Mexico Auger TLP pile template and piles have a three-coat paint system in addition to cathodic protection with aluminum/zinc/mercury-type anodes. The corrosion protection system was designed in accordance with National Association of Corrosion Engineers (NACE) RP-01-76 for a targeted fatigue life of 1,200 years (Ulbricht et al. 1994).

Foundation behavior monitoring

Long-term monitoring is required to detect changes in the structure's foundation that may affect the structure's life cycle performance. Periodic inspections and permanent instrumentation are needed to achieve monitoring programs. Offshore regulations list permanent instrumentation requirements

when (a) the foundation depends on an active operation such as drainage systems, (b) design conditions are different or unusual, and (c) foundation settlement, tilt, or penetration need to be monitored (Tjelta 1993).

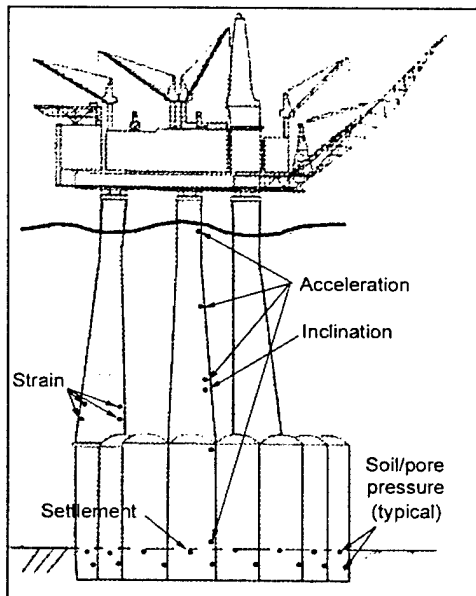


Figure 77. Foundation monitoring sensors on the Gullfaks C platform (after Tjelta 1993)

The skirted gravity-based foundations have a special need for long-term monitoring due to their reliance on active drainage inside the suction skirts. Filters placed on the inside walls of the skirts allow pumping to reduce pore-water pressures. Pressure transducers, accelerometers, settlement gauges, inclinometers, and/or strain gauges are permanently mounted at various locations on the foundations. Tjelta (1993) details the long-term monitoring instrumentation program for the North Sea's Gullfaks platform, shown in Figure 77.

The foundation monitoring system of the North Sea's Hutton TLP includes unique settlement monitoring gauges. Basically, manometer tubes were connected to both the

piling structure and the adjacent seabed. It was reported by Stock, Jardine, and McIntosh (1993) that the tubes contained pressure transducers capable of detecting differential displacements within an accuracy of 0.1 mm. The transducer outputs were acoustically transmitted to the surface, where settlements were recorded and integrated into the performance monitoring system.

Long-term monitoring of a pile foundation for a jacket platform was detailed by Sharp and Kenley (1993). Strain gauges and accelerometers mounted on the piles and pressure transducers placed beneath the mudmat were monitored beginning with the installation of the platform. Loading variations were recorded as a function of short-term events (such as high wave loading) and long-term performance.

6 Summary and Conclusions

This report provides a general overview and description of geotechnical foundations constructed in the wet (underwater). Case histories of selected (published) in-the-wet foundation projects were chosen. Since the largest quantity of in-the-wet foundations have been constructed offshore, most of the detailed information regarding foundation design and installation was taken from published offshore industry sources.

Three generalized underwater foundation types are described (improved-site, gravity-based, and pinned). Design, installation, and testing considerations are discussed for each foundation type. Site assessment principles remain the same as for onshore foundation sites, but the equipment technology for underwater site investigations may be more innovative and unique. For example, remotely controlled equipment is not commonly used for onshore site investigations. Sampling techniques and equipment may also differ from those used onshore.

Focus is placed on two of the most common pinned foundation types (pipe piles and drilled shafts) for design and installation considerations. The use of the suction pile, which has not been used for onshore applications, is also discussed. Foundation connections (interface between the foundation and structure) are not discussed since they are highly structure-specific.

Installation equipment and procedures are discussed, and differences between onshore and underwater foundation construction techniques are highlighted. Inspection and testing procedures are similar to those onshore, with the exception of pile load test frequency and configurations.

In conclusion, underwater foundations constructed in the wet may allow an alternative to expensive in-the-dry cofferdam construction. Underwater foundations have been used extensively offshore, since in-the-dry installations are not an option in deeper water and, consequently, much of the offshore expertise and technology developed over the last 50 years may be directly applied to the Corps of Engineers shallow (in-the-wet) sites. As the Corps and its contractors develop more experience regarding design and construction of in-the-wet foundations (especially for navigation and flood control structures on inland waterways), the innovative in-the-wet construction approach may offer a significantly beneficial alternative to typical in-the-dry construction.

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Appendix A

Underwater Foundation

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14. ABSTRACT <p>This report provides an overview and description of the design and construction of underwater geotechnical foundations and offers preliminary guidance based on past and current technology applications. Most of the state-of-the-art technology has been developed during 60 years of experience by the marine offshore industry solving complex foundation engineering challenges in the deep-ocean frontier, selected design firms and contractors, and, to a limited extent, local practice. Direct applications may or may not be made to site-specific underwater foundations in shallower rivers and inland waterways. However, most of the principles, techniques, and equipment are related.</p> <p>Case histories of selected in-the-wet (underwater) foundation projects are discussed for several types of structures: navigation, flood control, and dam structures; bridges; immersed tube tunnels and pipelines; offshore oil platform structures; and other underwater-founded structures.</p> <p>Three generalized underwater foundation types are described: improved-site, gravity-based, and pinned. Emphasis is placed on two of the most common pinned foundation types (piles and drilled shafts). Installation equipment and procedures are discussed, and differences between onshore and underwater foundation construction techniques are highlighted. Inspection and testing procedures are similar to those onshore, with the exception of pile load test frequency and methodologies.</p> <p>Underwater foundations constructed in-the-wet may allow a significantly beneficial alternative to the more expensive in-the-dry cofferdam construction, as experience is gained in applying this technology to the unique project needs of the U.S. Army Corps of Engineers.</p>														
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